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Repairing and Waterproofing the Nashville Water-Works Reservoir

By W. W. Southgate*

SYNOPSIS—A slid-out section of masonry wall was rebuilt, the interior face of the old and new wall was waterproofed with cement-gun plaster, the bottom of the basin was waterproofed with asphalt felt and then covered with concrete slabs, a flexible concrete hinged joint was inserted at the junction of the wall and floor and a drain was laid beneath the line of junction.

At midnight on Nov. 4, 1912, a break occurred in the wall of the east basin of the Nashville water-works reservoir. A section of wall 150 ft. long slipped on its foundation for about 30 ft. It became the duty of the writer, as City Engineer, to supervise the repairs under plans and specifications by Hering & Gregory, New York City. The work was successfully accomplished in the fall and winter of 1913 and in January, 1914, the basin was put back into service.†

In substance the plans and specifications provided for: (a) The restoration of the broken wall; (b) the waterproofing of the walls with cement-gun plaster; (c) the waterproofing of the floors by means of an asphalt-felt membrane which covered the entire floor and lapped up about three feet on the walls all around the interior of the basins; (d) the protection of the asphalt membrane with concrete slabs; (e) the construction of a

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†See "Engineering News," Nov. 4, 1912, for a description of the break, and Feb. 6, 1913, for an account of the scheme for repairs, with extracts from the specifications. The design and construction of the original reservoir were described in "Engineering News," June 15, 1888.

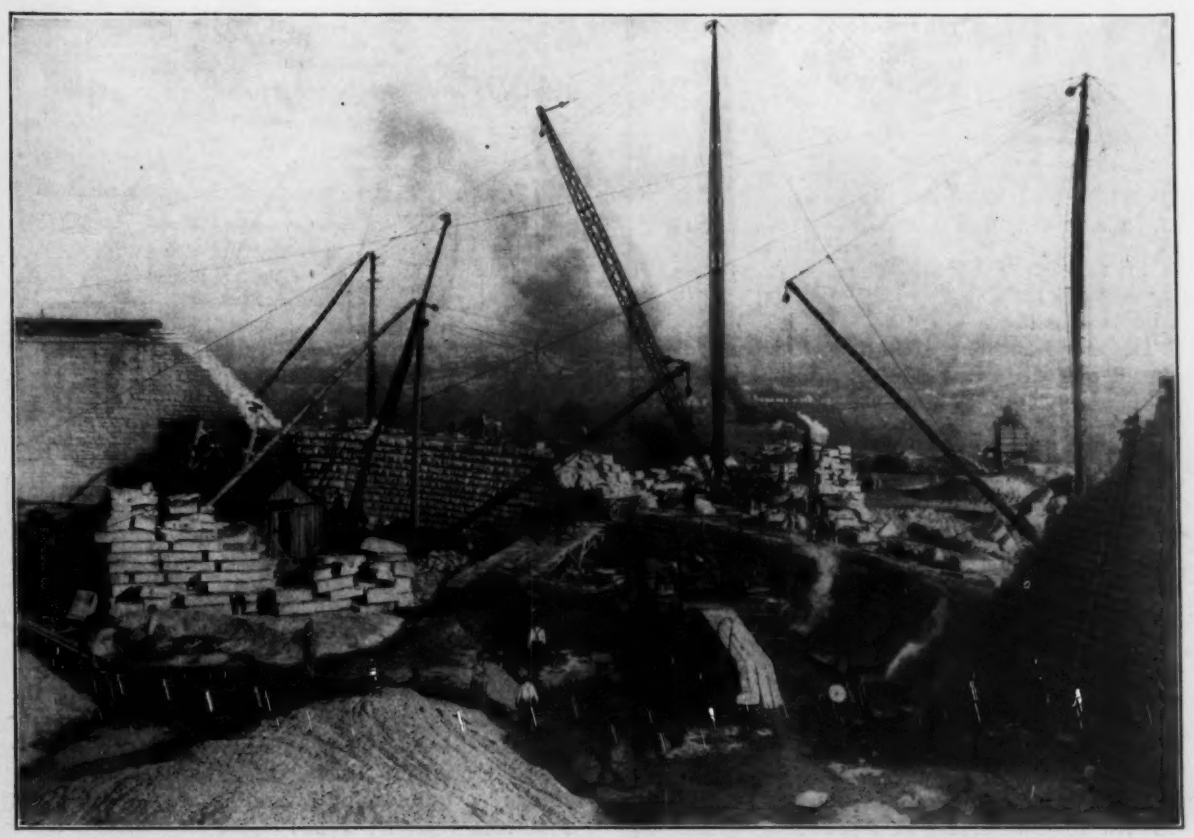


Fig. 1. Rebuilding the Breached Wall of the Nashville Water-Works Reservoir

flexible, concrete hinged joint all around the interior of the basin, between the walls and the superimposed concrete floor slabs, by means of wedge-shaped concrete blocks cast in place in the V-shaped trough formed for the purpose at the junction of the floor with the walls; and (*f*) the construction of a telltale clay-tile drain all around under the junction of the floor with the walls, and terminating in a manhole outside, so as to reveal any leakage that might occur in the floor or in its junction with the walls.

The walls of the reservoir (Figs. 1 and 2) were built of limestone rubble masonry, laid up in Louisville natural-cement mortar. There is a natural-cement concrete core about two feet in average thickness, extending from bottom to top, and midway of the section of the walls.

The old masonry was found to be in good condition, the fractures showing more generally in the stone than

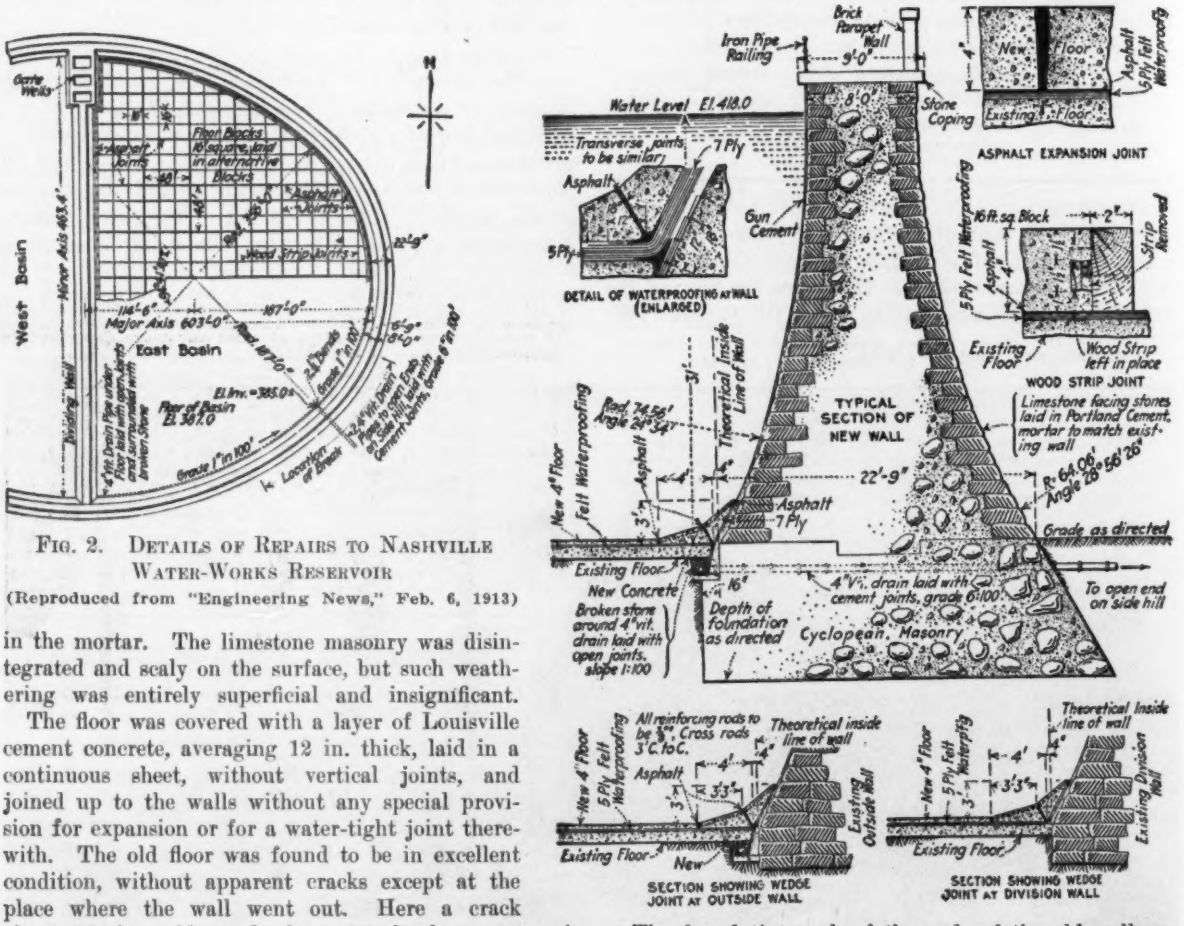

West Basin
East Basin
Major Axis 603'-0"
Minor Axis 463'-4"
Floor Blocks 18 square, laid in alternative Blocks
Wood Strip Joints
Asphalt Joints
Existing Wall
Pier of Basin El. 367.0
Grade 1" in 100'
Location of Breaks
Brick Parapet Wall
Iron Pipe Railing
Water Level El. 418.0
Stone Coping
Transverse joints to be similar
7 Ply
Asphalt
Gun Cement
5 Ply
DETAIL OF WATERPROOFING AT WALL (ENLARGED)
New Floor
Existing Floor
Asphalt 5 Ply Felt Waterproofing
ASPHALT EXPANSION JOINT
16 ft. sq. Block
Strip Removed
5 Ply Felt Waterproofing
Wood Strip left in place
WOOD STRIP JOINT
TYPICAL SECTION OF NEW WALL
Limestone facing stones laid in Portland Cement mortar to match existing wall
Theoretical Inside Line of Wall
Rad. 74.56' Angle 24° 54'
Rad. 64.05' Angle 25° 56' 26"
22'-9"
New 4" Floor
Felt Waterproofing
Grade as directed
Existing Floor
New Concrete
Broken stone around 4" tile drain laid with open joints, slope 1:100
Depth of foundation as directed
Cyclopean Masonry
4" V.t. drain laid with cement joints, grade 6:100
To open end on side hill
All reinforcing rods to be ⅜", Cross rods 3' C. to C.
Existing Outside Wall
SECTION SHOWING WEDGE JOINT AT OUTSIDE WALL
Existing Division Wall
SECTION SHOWING WEDGE JOINT AT DIVISION WALL

FIG. 2. DETAILS OF REPAIRS TO NASHVILLE WATER-WORKS RESERVOIR

(Reproduced from "Engineering News," Feb. 6, 1913)

in the mortar. The limestone masonry was disintegrated and scaly on the surface, but such weathering was entirely superficial and insignificant.

The floor was covered with a layer of Louisville cement concrete, averaging 12 in. thick, laid in a continuous sheet, without vertical joints, and joined up to the walls without any special provision for expansion or for a water-tight joint therewith. The old floor was found to be in excellent condition, without apparent cracks except at the place where the wall went out. Here a crack about ½ in. wide and about 15 ft. long, approximately parallel with, and about 1 ft. from, the wall and located across a geological fault, was revealed after the disaster.

REPAIR OF WALL—After clearing the site, excavation for the foundation for the restoration of the broken section of the wall was carried down to sound solid rock. Core drillings were made 8 to 10 ft. through the underlying rock to assure a perfectly stable foundation before laying the masonry. The clay and loose detritus were removed from the fault in the underlying formation down to good sound rock, a 6-in. tile subdrain was laid in the bottom of this excavation leading outside the foundation, and the pit was filled with rich concrete flush with the foundation rock upon which the new masonry was built. The ends of the intact walls were cut and raked from bottom to top, back to perfectly sound masonry, and carefully cleaned of old mortar and unsound stone by means of pneumatic chisels. As a precaution against cracks opening at the junction of the new masonry with the old, a flexible U-shaped, heavy, sheet-lead stop joint was inserted by cutting a dovetail groove in the concrete core from bottom to top of the ends of the old wall, and by anchoring one end of the lead joint therein with rich concrete in advance of the new masonry, and by leaving the other end free. The fold in the joint was protected with tar felt to assure free movement, and the new masonry was built around the free end thereof. This contrivance was simple, effective, of very little trouble, and inexpensive.

The foundation rock of the ends of the old wall was only about three feet below the original surface of the ground, and the underlying rock strata were comparatively thin and sandwiched between with clay seams. The pit for the new masonry was carried down vertically about 8 ft. below the bottom of the ends of the old wall for a solid rock base. This left the ends of the old wall tilted up at critical points upon a comparatively unstable foundation. This was effectually safeguarded by carefully excavating around and about 15 ft. back beyond the ends of the old masonry, down to the solid ledge upon which the new masonry was

founded, and by building a heavy buttress integral with, and tied into, the main new-masonry foundation by means of numerous 30-ft. railroad rails, thus hooking and overlapping the new-masonry foundation around and beyond the two ends of the foundation of the old wall. These buttresses were of about the same cross-section as the main new foundation and were brought up flush with the surface of the ground.

The new section of the wall was built of cyclopean masonry. The concrete used in lieu of mortar was a 1:3:5 mix for the foundation and a 1:2:4 mix for the masonry above the floor level. The facing stones for the wall were placed about one or two courses in advance and allowed to set before the backing was placed. The building of the masonry was practically continuous, day and night; there were few interruptions from rain, the season being a delightful Indian summer. When forced to stop the masonry, ample provision for bonding was made by leaving numerous stones projecting above the course then in process. Toothing of new masonry was not allowed, and bonding was made by breaking the new stones across over the top of the old masonry, which had been scrupulously cleaned to the naked stone. A superior quality of limestone, with good natural beds, was used in the masonry, and concrete was made of the same quality of limestone screenings, crushed rock that passed a 1-in. ring, and portland cement. The new masonry resulted in practically a water-tight job, and up to the present there has been no indication of an open joint or crack at the junction of the new masonry with the old. The length of the new section of masonry was about 150 ft. at the floor level and about 210 ft. under the coping.

Waterproofing the Interior Face of the Wall with the Cement Gun—The waterproofing of the interior face of the old wall with cement-gun plaster proceeded simultaneously with the building of the new masonry, as did also the waterproofing of the floor with the asphalt membrane. Portable scaffolds on wheels, leaning against the wall and so balanced as to be easily shifted, were provided for the workmen. An electrically driven air compressor, capable of delivering about 250 cu.ft. of air at 50 lb. per sq.in. per min., was installed about the center of the basin. Steel supply-piping of ample capacity for compressed air was laid from the compressor around the walls, with taps for the flexible air hose at convenient intervals. Similar supply-pipes for water for use with the cement gun and for spraying under air pressure were also provided. The walls were first thoroughly cleaned of all scale and foreign matter by means of pneumatic-hammer chisels, so as to afford sound stone faces for the plaster. By the same means the old mortar joints were gouged out to depths varying from 1 to 3 in. for the cement-gun mortar. The walls were then sand-blasted and sprayed immediately in advance of the cement gun, resulting in clean, sound stone faces and mortar joints.

The cement-gun plaster, composed of one volume of portland cement to three volumes of clean sand, followed right behind the sand blasting and spraying, before the walls could dry.

The whole interior of the walls, including the new masonry, was coated. The final result is an assured success. For a test the basin was first filled with perfectly clear water. A slight seepage near the top of the walls in isolated spots and clusters manifested itself. This disappeared when the water was lowered to a height of about 15 ft. Inspection of the plaster was made by means of a skiff. Some fine hair-line checks were discovered, evidently caused by contraction during the frosty weather in which the work was done. These were near the top of the wall, where the exposure was greatest. These places were carefully brushed over with neat-cement grout, the basin was again filled, and the seepage entirely disappeared, and there has been no recurrence of this trouble. The walls (Fig. 3) are manifestly water-tight.

Putting Asphalt Felt on the Floor—Before laying the asphalt-felt membrane, the old concrete floor was carefully cleaned and flushed off with a powerful stream, and all loose scale was removed. All rough places and sharp depressions were then flushed full and brought to a smooth plane with rich cement mortar. After thoroughly drying, the floor was well painted with asphalt cement, dissolved in naphtha for a priming coat. This was followed with a very heavy mop coat of asphalt cement heated to a temperature of about 325° F. The

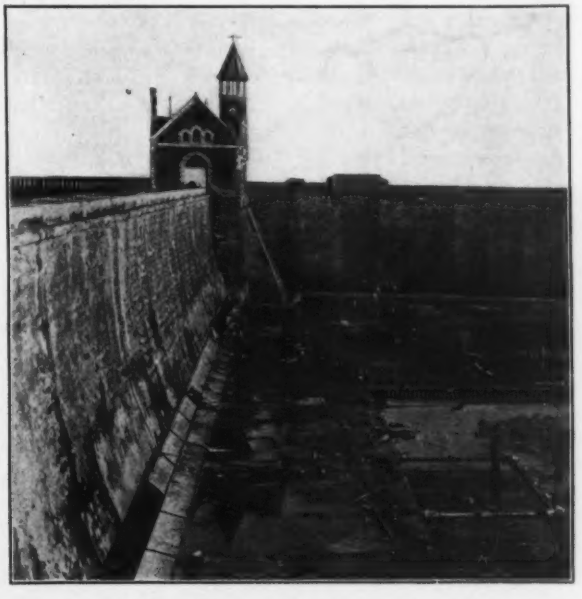


Fig. 3. Nashville Reservoir Wall after Having Been Plastered by Cement Gun

(The concrete wedge blocks and shoulders may be seen along the division wall at the left. The dark belt is the asphalt-felt membrane lapped up onto the wall. The basin at the left of the wall was nearly full of water when the view was taken, but showed practically no leakage.)

asphalt roofing felt followed closely behind the mop coat, in alternate layers of felt and heavy mop coats. Each layer of felt was carefully rolled down before the succeeding mop coat and next layer of felt, care being taken to squeeze out all the air bubbles. The asphalt felt overlapped and broke joints 3 in. on longitudinal edges and 10 or 12 in. on ends. Five layers of the felt were employed, ending with a heavy mop coat all over the top.

The cement-gun plaster was applied in an average thickness of 3 in. in a band about 3 ft. in height above the floor line all around the bottom of the walls, and was made continuous and carried down to the bottom of the

trench for the tile underdrain hereinafter described. Where this heavy plaster formed the wall side of the tile trench, it was floated and molded into a skewback for receiving one end of the concrete cover slab over the trench—which was cast in place, a corresponding skewback having been cut along the edge of the old concrete floor slab to receive the other end of the cover slab. During the same operation the cement-gun plaster was floated to a smooth even surface in a belt about 3 ft. wide all around the junction of the walls with the floor. A screeding strip of 7/8-in. pine flooring was secured in place all around the top edge of the heavy plaster belt, by shooting the mortar against the edges of and flush with the surface of this wooden strip. The latter served a twofold purpose: First, as a screed for floating the thick plaster belt, similar to a wainscoting, and secondly, as a nailing strip to tack the asphalt membrane to and to hold the latter in place until the wedge blocks were cast in place.

As indicated, the asphalt membrane was made continuous and lapped up on the walls in a seven-ply thickness.

Laying Concrete Floor Slabs—The 4-in. concrete cover slabs (Fig. 2) were cast upon the top of the asphalt membrane in alternate units 16 ft. square. After setting, the succeeding units were cast in the blank spaces in the usual way. Contiguous units were separated by 1/2x1-in. dovetail-shaped soft cypress strips, which were tacked to the 2x4-in. wooden forms with sixpenny nails, just sufficiently to hold the cypress strip in place against the form, and leaving the head and greater part of the nails projecting into the concrete. After the slab had properly set, the 2x4-in. forms were carefully removed, and the cypress strip remained in place for keying and calking the joints between the slabs. Around every cluster of nine 16-ft. units, or 48-ft. square, asphalt-cement expansion joints were poured, 3/8 in. thick at the bottom and 5/8 in. at the top.

Concrete Wedge Blocks—For a bearing for the concrete wedge blocks (Figs. 2 and 3) the horizontal shoulder blocks were first cast in position on top of the asphalt membrane, leaving the V-shaped trough between the shoulder blocks and the wainscoting and asphalt membrane thereon, around the walls, into which the flexible wedge blocks were molded. Before casting the wedge blocks, the shoulder blocks were backed up by casting all adjacent floor slabs, and all allowed to properly set. Making the asphalt joint between the shoulder and the wedge blocks was a simple process. Flooring strips were laid down parallel on the concrete floor slabs, of the proper widths and lengths to serve as a mold. A strip of flooring felt was laid in the bottom of this mold, upon which the hot asphalt was poured and allowed to cool. When sufficiently set, the flooring strips were removed and the resulting ribbon of flexible asphalt was easily picked up and laid in place in the V-trough against the concrete shoulder blocks. Hot asphalt to the depth of about 2 in. having been poured into the bottom or apex of the trough and allowed to stiffen, the wedge blocks were then cast in place, the asphalt membrane on the wainscoting having first received its final mop coat. The concrete shoulder and wedge blocks were each reinforced with steel rods and separated at the ends by asphalt expansion joints, by a simple contrivance. The mechanical action of the wedge blocks was instantaneous, as was at once revealed by the asphalt being slightly squeezed out of the joints between the shoulder and the wedges. The asphalt membrane was then cut loose from the flooring screed strips, whereupon the latter were easily removed. The small groove left thereby along the top edges of the wedge blocks was poured full and sealed with hot asphalt.

This flexible wedge-block contrivance is quite ingenious, and in the writer's opinion is an entirely efficacious method of providing an automatic seal and water-tight joint between the concrete floor and walls of the reservoir.

The Underdrain—A trench about 2 ft. wide, and averaging 3 ft. for the underdrain, was excavated down well below the bottom of the old concrete floor, all around beneath the junction of the floor with the outside wall of the basin. This trench was cut through the underlying rock ledges by means of hammer and bull points, blasting not being allowed. The bottom of the trench was covered with a 6-in. slab of rich concrete, to a grade with a fall of 0.1 in. per 100 ft. This slab was carefully cleaned and the cement-gun plaster was shot and faced down to a tight junction therewith and molded into a skewback, as described for the cover slab. The other face of the trench was left raw, so as to allow a vent to any seepage through the floor into the underdrain. Hard-burned clay tiling was laid with open joints on top of the bottom slab and packed with 1-in. crushed stone from which all dust and screenings were excluded. The crushed-stone packing was brought up to the bottom of the old concrete floor slab and covered with a sheet of roofing felt. The cover slab was then cast in place in the skewbacks above described, on top of the felt, and finished flush with the top of the old concrete floor.

The underdrains lead from both directions around the junction of the floor and walls to a common manhole outside the wall, through which both drains were carried in the new masonry. The telltale drains are examined frequently in this manhole. The greatest amount of leakage thus far revealed is at the rate of thirty drops of water per minute, and this has been practically constant since the basin was put into service.

This demonstrates that a practically water-tight job was accomplished both in the floor treatment and in its junction with the walls.

The total cost of the work was $101,324. The E. T. Lewis Co., of Nashville, was contractor for the entire work. The General Cement Gun Co., Chicago, was subcontractor for waterproofing the walls with cement-gun plaster. The St. Louis Roofing Co., St. Louis, Mo., was subcontractor for the asphalt waterproofing of the floor. Dow & Smith, New York City, made the chemical analyses and tests of the asphalt cement and roofing felt. Speed portland cement, made by the Louisville Cement Co., was used in the masonry, in the concrete, and in the cement-gun plaster. The cement tests were made at the factory, and certified by Pierce Butler, Louisville, Ky.

Lining the West Basin Walls—The west basin of the same reservoir was, during the summer of 1914, waterproofed in a similar, but not so elaborate, manner. The commission form of government having been inaugurated in the meantime, the work on the west basin was done under the direction of the commissioner of water-works.

Water Coagulation, Sedimentation and Aeration Plant at Norristown, Penn.

By S. Cameron Corson*

A reinforced-concrete coagulation and sedimentation basin with superimposed aërators has recently been put in operation by the Norristown (Penn.) Insurance & Water Co. to treat Schuylkill River water preliminary to filtration. Aëration is designed to remove tastes and odors from gas-plant wastes discharged into the river above the city. The filters are of the Jewell type and were installed 12 years ago.

Water is taken from a crib in the Schuylkill River through 1800 ft. of pipe to low-lift pumps. A coagulant is added to the river water in the force main. The latter discharges through a hydraulic control valve into an influent or receiving chamber (Fig. 1) and thence through sluice gates to the two aërator units, each of which is arranged to discharge into its corresponding compartment of the sedimentation basin. Each unit consists of an inclined plane having an area of 230 sq.ft., covered with iron plates on which are cast projections (Figs. 1 and 2) arranged to break up the water and secure its intimate

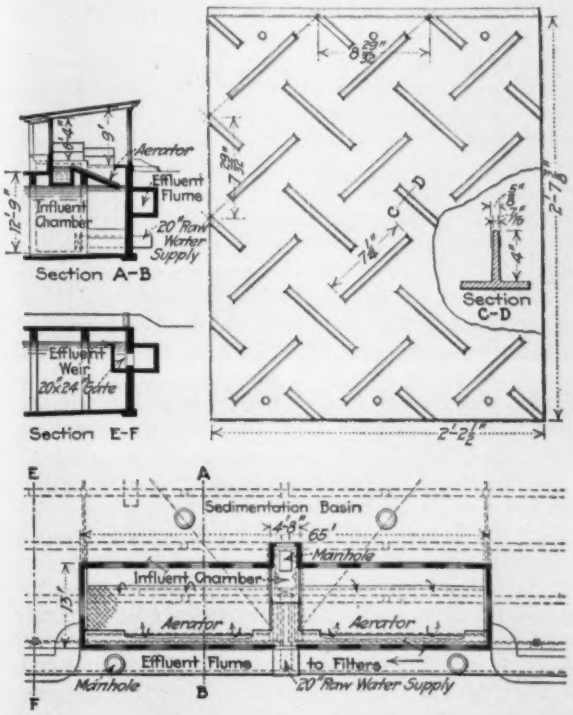


Fig. 1. Details of Norristown Water Aërator

contact with air.† The windows shown in the view provide ample means for ventilating the aërator house.

Each compartment of the sedimentation basin is 129 ft. long and 63 ft. wide. When filled to an average depth of 10 ft. the whole basin has a storage capacity of 1,200,000 gal. When supplying the filters at a rate of 5,250,000 gal. per 24 hr., the basins provide a 5½-hr. period of coagulation and sedimentation and the average velocity of the water passing through the basin is about 0.8 ft. per min. Baffle walls cause the water to pass twice the length of the basin before discharging over the effluent weirs (Fig. 1) on its way to the filters. The floors of the basin slope

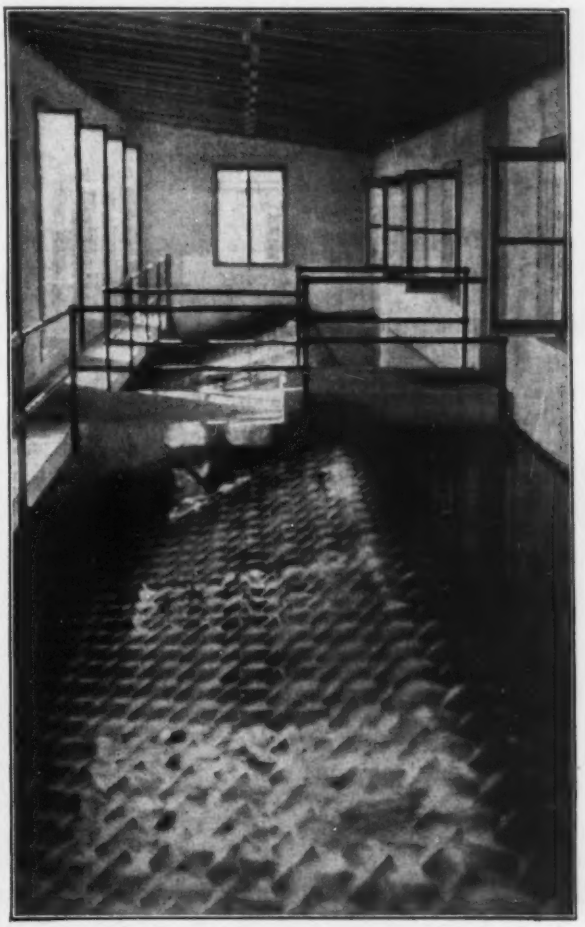


Fig. 2. View of Norristown Aërator

to clean-out valves, from which sediment pipes lead to the river. The sediment is pushed to the pipes by hand and by flushing. An overflow pipe for each compartment is connected to the drainage system.

The hydraulic influent regulator already mentioned is a specially constructed valve operated by a float and pilot-valve mechanism arranged to maintain an approximately constant level of water in the basin.

The filters are located in a nearby building. They consist of nine units, each 16 ft. in diameter and 16 ft. in depth. The filtered water drops to a clear-water basin, 48x100 ft. in plan and 9 ft. deep, directly beneath the filters. From this basin the water is pumped to the main distributing reservoirs, which are located on the highest ground in the city.

The new aërator and sedimentation basin were constructed by the New York Continental Jewell Filtration Co., of New York, under the direction of Robert A. Jackson, Superintendent of the Norristown Insurance & Water Co. The cost of the work, which was largely in rock excavation, was about $40,000.

*Borough Surveyor, Norristown, Penn.

†We are informed that this method of aëration has been successfully used for a number of years at Newport, R. I., and Lexington, Ky., to remove taste and odors due to algæ.—EDITOR.

Operations of the Cincinnati Water-Filtration Plant for 1914*

By J. W. Ellms†

SYNOPSIS—Nineteen billion gallons of water was pumped from the Ohio River to large settling reservoirs, coagulated with sulphate of iron and lime and passed through mechanical filters. During the first five months of the year the water was also disinfected with liquid chlorine. The two principal objects of treatment were the reduction of bacteria and of turbidity.

The two settling reservoirs were in continuous operation throughout the year. On account of the smaller amount of suspended matter carried by the Ohio River water during the year, as compared with the preceding six years, these reservoirs have received less sediment than usual. Their efficiency in removing turbidity, however, has been greater than ever before, as 74.1% of the sediment-producing turbidity has been removed. This is a removal 9% greater than the highest recorded yearly average since the plant was started.

The average turbidity of the Ohio River water was only 120 parts per million as compared with 204 p.p.m. for an average of the preceding six years. The average turbidity was therefore over 41% less than usual. For 63% of the year 1914 the turbidity of the river water was under 100 p.p.m., while for 34% of the time it was between 100 and 500 p.p.m. It is of interest to note that the median turbidity of the river water was but 60 p.p.m., or one-half of the yearly average.

The greater ratio than usual of ground water to surface water in the Ohio River during the year was shown by the mineral constituents being higher than was ever before recorded. For example, the total hardness was 93 p.p.m., and the chlorine 24.1 p.p.m., or nearly 30% greater in hardness and 42.6% greater in chlorine than the average results for the six years preceding 1914.

Coagulation Basins

These basins were cleaned in April and again in October. All three basins have been in continuous use except for the two days' time required to clean each of them.

The water entering the coagulation basins contained on an average 31 p.p.m. of turbidity, which is 62% less than the average for the preceding six years. Of this sediment the basins removed 21.7%, which is the lowest recorded average since the plant was started. The basins delivered to the filters a water which had on an average but 5 p.p.m. of turbidity, or only one-fourth the usual amount of turbidity applied to the filters.

There were used during the year for coagulation purposes 1630.45 tons of sulphate of iron and 907.45 tons of lime. These chemicals were applied to 19,154,935,223 gal. of settled water. The average amount of sulphate of iron applied was 20.4 p.p.m., or 170 lb. per million gallons; the lime applied averaged 11.4 p.p.m., or 95 lb. per million gallons.

Filters

Nearly 0.7% more water was filtered in 1914 than in 1913, and about 1.1% more filtered water was delivered into the service. The percentage of wash water used for cleaning filters was 2.24, as compared with 2.65 in 1913. The average period of service of the filters was 19.5 hours, or 2.24 hours longer than for the preceding year. This was due principally to somewhat less trouble than usual from microscopic plant growths in the river and settled water during the year 1914.

The hardness of the filtered water averaged 106 p.p.m. This average hardness is nearly 26.2% greater than the average of the six years preceding, and is accounted for, as previously noted, by the long-continued low stages of the water in the Ohio River, which increased the ratio of ground to surface water. The filtered water was harder on an average than the river water by 13 p.p.m. or 0.76 grain per gallon.

The sanitary quality of the water has been maintained at the high standard reached in previous years. On an average the filtered water contained 36 bacteria per c.c. As the river water contained more bacteria on an average (16,500 per c.c.) than has ever been obtained before, the proportion removed (99.8%) has been satisfactory. Although the average number of bacteria in the Ohio River water was 16,500 per c.c., the median number was 2900. The average and median numbers of bacteria in the filtered water were 36 and 20 per c.c. respectively.

There was used during the first five months of the year for disinfection purposes, 5,998.5 lb. of liquid chlorine, equal to 0.1 p.p.m. or 0.82 lb. per million gallons. This chemical was applied to the filtered water, and reduced the number of bacteria on an average, while being applied, 83.6%.

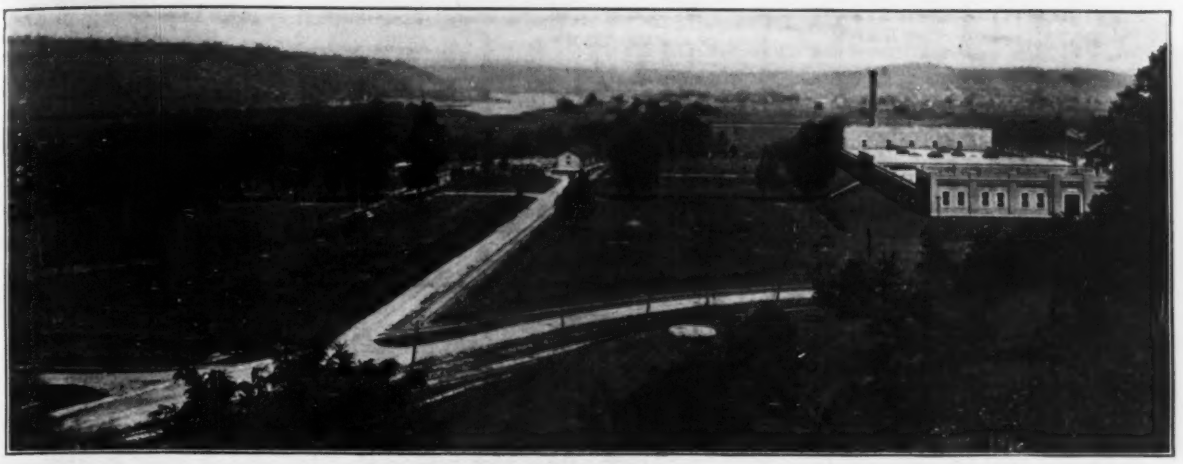
For 43.75% of the time the number of bacteria in the river water was below 1000 per c.c., and for 54.54% of the time, it was between 1000 and 100,000 per c.c. During 95% of the time the filtered water had less than 100 bacteria per c.c., while for 4.4% of the time the number was less than 250 per c.c., and during only 0.6% of the time was it over this number.

The B. coli index shows that approximately 15,000 bacilli of this species were present in every liter of river water, while only 2 or 3 could be found in the same amount of filtered water.

The purity of the water-supply of the city is confirmed by the very low death rate from typhoid fever during the year. There were 23 deaths from this disease during the year, or a rate of 5.71 deaths per 100,000 of population. In 70% of the cases in which death resulted the disease had been contracted outside of the city, or, if within the city, a source of infection had been fairly well established which eliminated the possibility of the public water-supply being the cause. In the remaining seven cases (30%) where death occurred, the source of infection was unknown.

*Full text of the official report for the year, reprinted from an advance copy. Appended to the report are 12 tables, some of which will be given in a later issue.

†Superintendent of Filtration, Cincinnati, Ohio.



FAR, NEAR AND INTERIOR VIEWS OF THE CINCINNATI FILTRATION PLANT

Filtered-Water Reservoir

This open reservoir has developed the usual amount of microscopic plant growths, which attach themselves to the bottom and sides of the basin. No objectionable odors or tastes have resulted from their presence in the water.

Laboratories

In the laboratories a large number of samples of water have been examined, besides many other substances used in this or other divisions of the water department. In the chemical laboratory 4862 samples of water have been examined, involving 10,150 different determinations. In addition, 104 samples of water were subjected to a complete sanitary chemical analysis and 24 samples to a mineral analysis. Besides the above work with water, there have been 354 samples analyzed, consisting of coal, lime, sulphate of iron, asphalts, pitches, sand and other miscellaneous material.

In the bacteriological laboratory 4848 regular samples of water have been examined for the number of bacteria which they contained, and 370 special samples. The tests made for B. coli communis number 5957. Six microscopical examinations have been made of samples of water from different parts of the purification system.

Cost of Operation and Maintenance

The total cost of operating and maintaining the filtration plant was $62,776 or $3.31 per million gallons of filtered water delivered into the service. Of this unit cost, $2.94 was for operation and $0.37 for maintenance. These are the lowest yearly costs yet obtained here.

There was expended for coagulating chemicals the sum of $22,536, or $1.19 per million gallons of filtered water delivered into the service. This is $0.48 per million gallons less for coagulating chemicals than in the preceding year, and accounts for nearly 80% of the difference in the unit costs for 1913 and 1914. In addition to the above sum for coagulating chemicals, there was expended $607 for disinfection with liquid chlorine, or $0.086 per million gallons of filtered water delivered into the service, which had been treated with chlorine.

COST PER MILLION GALLONS OF FILTERED WATER DELIVERED INTO THE SERVICE

Year	1908*	1909*	1910	1911	1912	1913	1914
Operation	$4.19	$4.17	$3.91	$3.77	$3.46	$3.44	$2.94
Maintenance	0.05	0.09	0.28	0.35	0.38	0.48	0.37
Total	$4.24	$4.26	$4.19	$4.12	$3.84	$3.92	$3.31
Chemicals†	$1.72	$1.89	$1.93	$1.86	$1.78	$1.67	$1.19

*The cost figures for 1908 and 1909 have been recalculated to agree with the method used since 1909, and so as to make results for the different years strictly comparable.

†Included in the total costs for each year.

Use of Liquid Chlorine at Buffalo Water-Works Intake

By H. F. Wagner*

The City of Buffalo has been drawing her water-supply for about three years from the new intake crib† situated at the head of the Niagara River, 6560 ft. from the new pumping station at Porter Ave. and 10,740 ft. as the tunnel runs to the old reconstructed station at the foot of Massachusetts Ave. Before this period the supply had been drawn from an old crib located in the Niagara River about 1000 ft. from the shore. The typhoid death rate had persisted at a high figure (24 per 100,000 average) and ice troubles prevented always drawing a sufficient supply. Therefore, on the advice of experts it was decided to build a new crib. Its location was such that interference from ice was entirely done away with and a water obtained as good as could be drawn from a body the size of Lake Erie.

But the fleets of freight and passenger boats have grown, and contamination of the lake has continued to increase. The main ship channel leading from Buffalo Harbor to the lake is but 3000 ft. from the new crib. This distance is so short that polluting matter from these vessels may easily find its way into the city's supply. In the winter time, after ice has formed, hundreds of fishermen go out on the ice. The danger arising from excreta which these people leave upon the ice is by no means to be neglected, for when the ice breaks up it is all carried in the direction of the crib.

The water-supply gave positive tests for colon bacilli in 1 c.c. during the greater part of the year, although there were periods after the ice had closed in when no colon bacillus was indicated in 5 c.c. On account of this condition, Col. Francis G. Ward, Commissioner of Public Works, and Henry L. Lyon, Deputy Water Commissioner, decided to install a disinfecting plant.* It was finally decided to use liquid chlorine and to introduce it at the intake-crib pier. Considerations of economy, space required, simplicity of apparatus and possible taste imparted to the supply affected the decision not to use hypochlorite. It was thought that the long run from the intake to the consumers' service lines would afford the best possible opportunity for complete admixture.

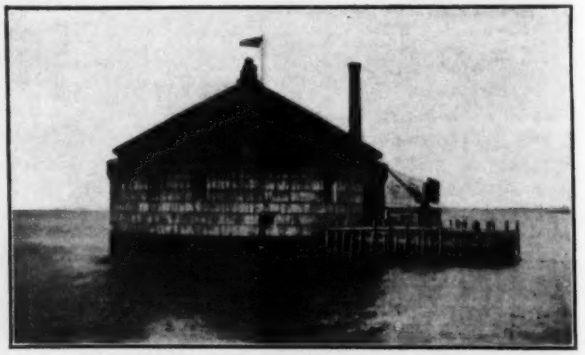


Fig. 1. The Buffalo Intake Pier

The solubility of chlorine is marked, particularly at lower temperatures; the deep yellow color of the solution gradually fades (rapidly in sunlight) owing to combination with hydrogen and liberation of oxygen. This action gives chlorine its bactericidal powers. Hypochlorite gives some excellent results but proper manipulation is difficult and considerable room is required. About two-thirds of the material (and some chlorine) is lost in a lime sludge. Amounts that will do effective disinfection and impart objectionable taste are not far removed.

The gas is introduced by means of a specially constructed apparatus developed by the Electro Bleaching Gas Co., of New York City, to meet the conditions at Buffalo. It is located in a room which is kept at a temperature of about 95° F., and consists essentially of (1)

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†Described in "Engineering News," Feb. 9, 1911.

*Chlorination was recommended by Rudolph Hering, as consulting engineer; alternate proposals for use of hypochlorite and liquid chlorine were suggested. (See "Engineering News," Feb. 12, 1914, p. 375.)

a heating cabinet for holding eight cylinders at a time and (2) duplicate absorption towers in which a water solution of the gas is made for discharge into the water-supply.

The heating cabinet is divided into two compartments, each holding a set of four cylinders, which are used al-

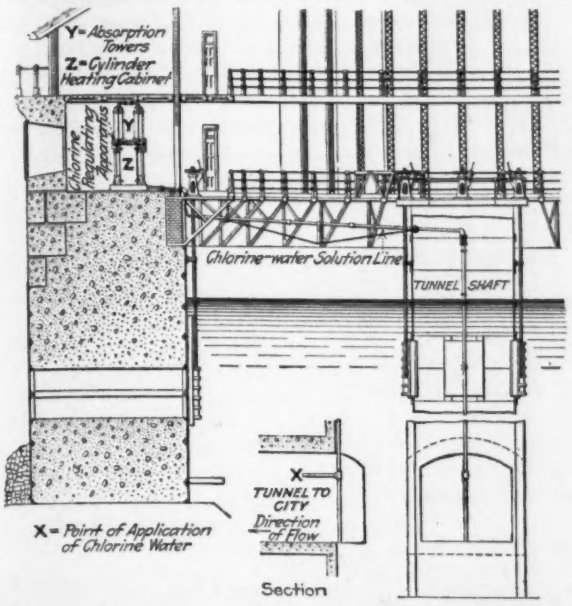


FIG. 2. ARRANGEMENT OF LIQUID-CHLORINE APPARATUS AT BUFFALO INTAKE

ternately. The liquid gasifies and escapes at once upon opening the hand valve; owing to the absorption of heat in vaporization the remaining liquid in the cylinders is gradually cooled, and the pressure drops until the required supply is no longer delivered. Therefore, a high room temperature is required and it is further supplemented by steam coils located in the heating cabinet close to the bottom of the cylinders where the gas is naturally coolest. By this arrangement it is possible to draw from one set of four cylinders for several hours at a time without much variation in pressure. When the pressure drops to the limit set the four reserve cylinders are switched in and the others given an opportunity to warm up, when pressure will again be exerted.

Connected to the heating cabinet and leading to the absorption towers are copper coils (Fig. 3), which conduct the gas through a regulating device, graduated to deliver in pounds per hour, which reduces the initial pressure to any desired value. The large connecting coils are used because the heating cabinet and cylinders are placed upon a set of scales (so that independent check may be made upon the delivering device at all times). After leaving the regulating orifice the gas passes through a ½-in. hard-rubber pipe and is carried up over the top and down the inside of the absorption towers and there discharged. The absorption towers (Fig. 3) are 12 ft. high and 14 in. in diameter and are made of a fiber preparation that withstands the corrosive action of the gas. Around the ½-in. pipe and filling the towers is coarse pumice stone. Over this percolates a continuous flow of water delivered into the top of the tower in the form of a spray and led by gravity from an elevated storage tank. This water, flowing downward, absorbs the chlorine gas forced in from the discharge pipe; the chlorine water is drawn off at the bottom by a 1½-in. hard-rubber pipe and conducted to a shaft (Fig. 2) in the center of the intake, where it is dropped vertically downward to the mouth of a 12-ft.

TYPICAL RESULTS OF CHLORINATION AT BUFFALO

Date	Raw Water Total Bacteria	Raw Water B. Coli	Treated Water Total Bacteria	Treated Water B. Coli	Chlorine Dose Parts per Million	Turbidity Average	
Dec. 16, 1914..	860	+	56	—	0.29	...	Fall
Dec. 17, 1914..	840	+	19	—	0.29	110	con-
Dec. 18, 1914..	1260	+	34	—	0.27	...	ditions
Feb. 22, 1915..	1130	+	11	—	0.17	...	
Feb. 23, 1915..	2060	+	14	—	0.19	15	Winter
Feb. 24, 1915..	1180	+	3	—	0.19	...	con-
Feb. 25, 1015..	420	+	5	—	0.13	...	ditions

tunnel, 40 ft. below the surface, which carries the water-supply to the pumping station. At this point the pipe makes a 90-deg. turn and extends 4 ft. into the tunnel where the disinfecting liquid is delivered. The hard-rubber pipe is inclosed throughout its entire length by a cast-iron pipe to protect it from possible injury. Both pipe line and absorption tower have been installed in duplicate, so that if repairs are required to one the chlorination process is not interfered with.

The results have proved satisfactory in all respects. Tests show no trace of free chlorine in the mains, and no tastes or odors have resulted from its use—not even during the periods of highest turbidity when the largest dose was being applied.

The total bacteria count fluctuates from less than 100 per c.c. to as high as 4000 and 5000 per c.c. on 20° agar, the latter count generally being obtained during the periods when the lake is disturbed by heavy gales and with the turbidity running as high as 125 and 130 parts per million. The highest dose of chlorine that has been delivered is 0.36 part per million. While complete efficiency is not aimed for, we do entirely eliminate all evidences of pathogenic organisms and obtain a very

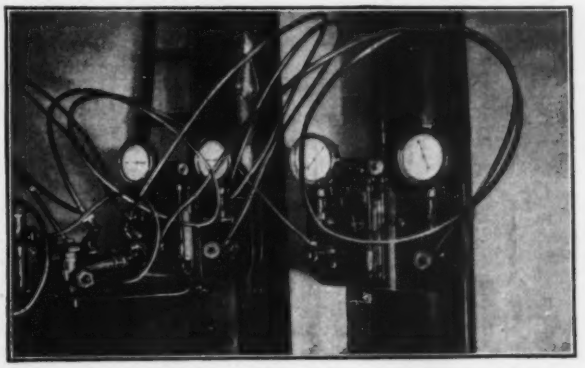


FIG. 3. CONNECTIONS FROM LIQUID-CHLORINE TANK CABINET TO ABSORPTION TOWERS

high reduction in the total count, as shown in the accompanying table.

A 100-per cent. efficiency is not hoped for, since our tests show that the organisms escaping are harmless and because complete sterility is secured only at needless extra expense. We could produce a sterile water, however, without making the water taste (which is one of the great advantages of liquid chlorine over hyprochlorite).

The typhoid death rate was decreased to 13 per 100,000 after the old intake was discarded; the chlorination plant has not been in use long enough yet to determine its influence on death rate, but a marked reduction is confidently looked for.

Stripping Water-Works Reservoirs

By Allen Hazen* and Geo. C. Whipple*

SYNOPSIS—Logarithmic probability paper has been used to plot the growth of organisms in stripped and unstripped reservoirs. The stripped reservoirs show no advantage over the unstripped.

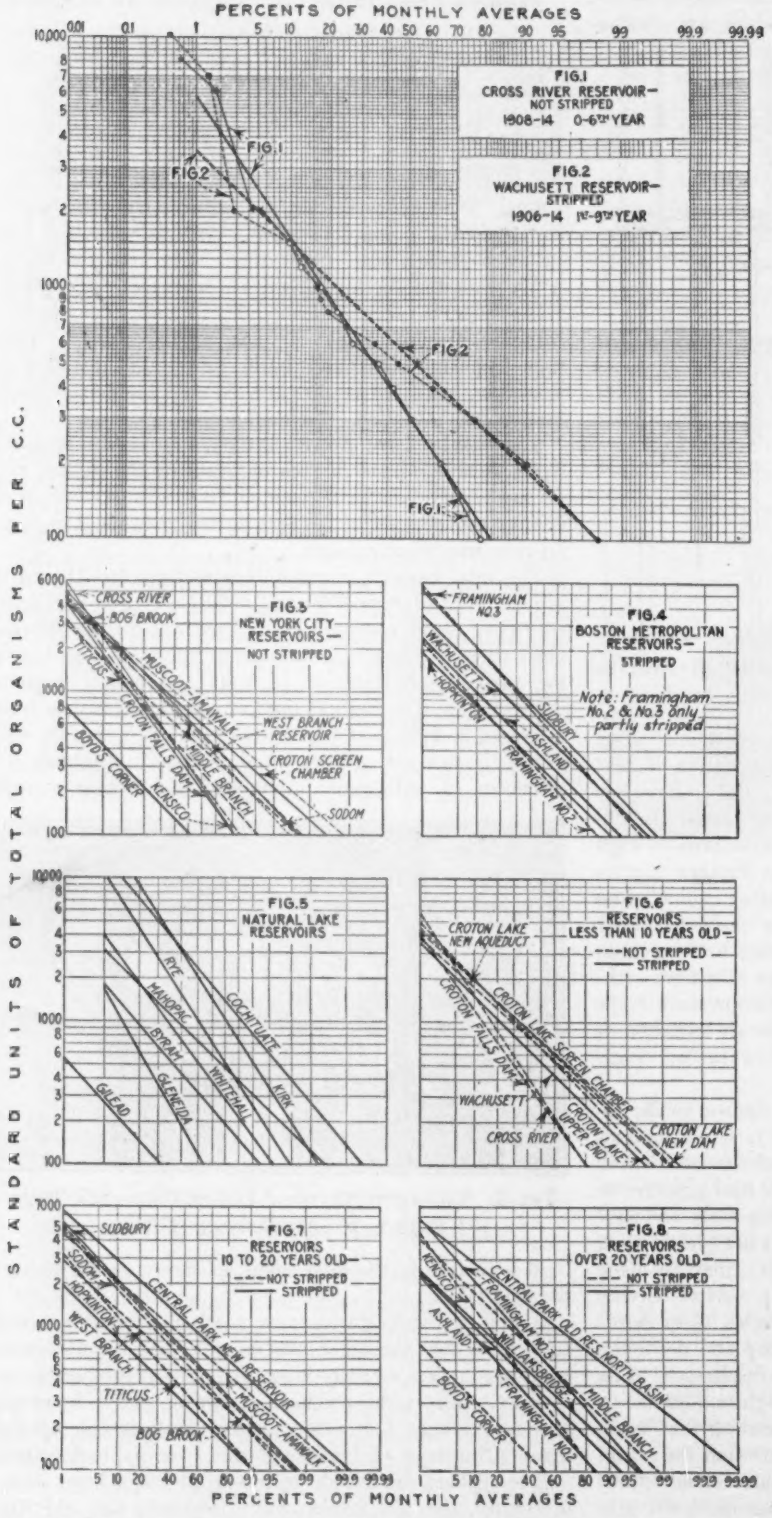
Nine years ago, when the plans for the Ashokan and Kensico reservoirs of the Catskill water-supply system for the City of New York were under consideration, the question was raised as to whether the soil should be stripped from their bottoms, as had been done for Boston and a few other cities.

This matter was exhaustively studied by George W. Fuller and the writers, with results adverse to stripping.† These reservoirs have since been built with only such cleaning as has been customary for reservoirs for public water-supply.

At the time of this investigation the Wachusett Reservoir, the largest and most important of all the stripped reservoirs, had just been completed and was being filled. The quality of water that it would produce had not been ascertained.

During the past year the study of nine years ago has been brought up to date. The data are now more complete, and it has been possible to compare the records of the Wachusett Reservoir with those of the New York system of about the same age that have not been stripped. In making this comparison a compilation of all available records of organisms in the several waters was made and a novel method was used.

It is well known that in games of chance most events tend to cluster around a certain median result and that the various phenomena of nature do the same. In the measurement of the lengths of leaves or the height or weight of men many more observations will be near the mean result than those which are at variance with the mean, and the frequency of departures of given amounts will follow in a general way this law of chance. It has already



GROWTHS OF ORGANISMS IN STRIPPED AND UNSTRIPPED RESERVOIRS

(The originals of each of the eight diagrams were plotted on logarithmic probability paper, the ruled portions of which were about 9¼ x 6½ in. Fig. 1 is a complete reproduction of the original, with the plottings of Fig. 2 superimposed. The originals of the other six diagrams have been clipped of blank portions, intermediate ruling omitted and the whole given a heavier reduction in scale than Fig. 1 and 2.—EDITOR)

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† "Engineering News," Vol. 57, p. 8.

been found that rainfall and stream flow may be considered as chance phenomena, meaning by chance, not that the phenomena occur without cause, but that so many causes are at work that the influence of each cannot be determined. The probability method may be considered, therefore, as applicable to phenomena where there are many inseparable causes at work.

An interesting use of probability methods is in connection with the occurrence of microscopic organisms in reservoirs. It is well known that many factors influence the growth of these organisms: Their food supply—nitrogen, carbonic acid, oxygen, hydrogen, phosphorus, sulphur, silica and iron, manganese, calcium, magnesium, etc.; the temperature and movement of the water, the penetration of light, the size, shape, depth and orientation of the reservoir, the nature of the soil at the bottom, the cleanliness of the catchment area, and others. Taken individually, we have some idea of how the different factors act, but no one knows how to combine them so as to obtain an adequate idea of the sum total of the influences of all. By plotting on probability paper the data showing the occurrence of organisms in a reservoir, some idea may be obtained as to the combined effect of the many factors which cause the organism to develop and the frequency with which growths of different magnitude are likely to appear.

It is not to be expected that such a method will be precise, for, as a matter of fact, there is not one law of frequency, but many. With certain data it is found that the plotted points fall near a straight line if the paper is drawn with probability intervals as abscissas and direct observations as ordinates. Rainfall records plot out in this way. In other cases a straight line is obtained by plotting on a paper drawn with probability intervals as abscissas and the logarithms of the observations as ordinates. This is the case with the microscopic organisms. Usually, one or the other of these papers serves fairly well.

In previous studies of this kind the organisms had been plotted chronologically. This showed the seasonal fluctuations and the fluctuations from year to year in each reservoir studied. The fluctuations were usually so great and so irregular that it was difficult to compare the results of different reservoirs with one another. What was needed was some method by which all of the results could be depicted on the same sheet. Comparison of the average results for the different reservoirs did not suffice, inasmuch as the same average figure might represent a uniform occurrence of organisms in moderate numbers or a combination of very high and very low results. The use of the probability paper took these facts into account.

The particular reservoirs studied were those of the water-supplies of the cities of Boston and New York. The results of the former were obtained from the laboratory records of the Metropolitan Water Board and those of the latter from the records of the laboratories of the Department of Water Supply, Gas and Electricity, of New York City.

The method of plotting was as follows: The results for each reservoir for each month from 1906 to date were arranged in the order of magnitude and plotted on logarithmic probability paper, the highest result being plotted at the left-hand side of the sheet. In order to save work the observations which were of about the same magnitude were combined into groups and the mean of the group plotted. Where the observations were scattered in the case of very high or very low results each individual observation was given a position on the sheet. The method will be best understood by examining Fig. 1, which shows the results for the Cross River Reservoir of the New York supply. On this sheet the crosses at the left indicate the five highest individual counts that were made in a nine-year period lower down. Each circle represents several determinations and in some cases as many as ten or twenty. The circles, therefore, were given greater weight than the crosses in determining the general slope of the line. After plotting the data in this way it was found that the results fall nearly in a straight line and a heavy line drawn through them was taken to represent the record of this reservoir. The fine lines merely connected the different points. The heavy line shows directly the percentage of observation exceeding any given limit. For instance, in Fig. 1 5% of the observations taken from the heavy line exceeded 2500 per c.c., 10% exceeded 1550, 50% exceeded 300 and 80% exceeded 105.

In some cases the individual plottings deviated from a straight line. This is shown in Fig. 2, representing the record for the Wachusett Reservoir. There was, however, no difficulty in drawing a straight line that represents all data reasonably well.

The Wachusett Reservoir (Fig. 2) was put in use about the same time as the Cross River and is important as being the largest and most recent stripped reservoir that has been built. It is much larger than the Cross River, but in other respects the two are comparable.

The straight lines representing all the reservoirs of the New York water-supply are grouped in Fig. 3. They include the main storage reservoirs of the Croton system and also the Kensico reservoir of the Bronx system. The distributing reservoirs, which are paved or lined with concrete, are not included. Fig. 4 shows in the same way all of the stripped reservoirs in the Metropolitan System of Boston. Fig. 5 shows the organisms in the natural lakes in both systems.

By comparing Figs. 3 and 4 it will be seen that the position and range of the lines are substantially the same for both the Boston and the New York systems. In other words, the stripped reservoirs of the Boston system contained about as many organisms and with about the same degree of frequency as the unstripped reservoirs of the New York system. High counts have been observed at times in both series of reservoirs. So far as these results go they do not indicate any change in favor of soil stripping as preventive of microscopic growths.

Fig. 5 shows that the organisms in natural lakes cover a wider range than in the reservoirs, whether stripped or unstripped. If a mean should be struck, the natural lakes would be found to differ only slightly from the stripped or unstripped reservoirs.

The records for the various reservoirs have been combined in other ways. For example, when the reservoirs less than ten years old were grouped by themselves in Fig. 6, it was found that the Wachusett, which was the only unstripped reservoir of that age, had a slightly smaller number of organisms than the newer reservoirs of the Croton system, which were not stripped. The difference, however, was not very great. In the reservoirs between ten and twenty years old, shown in Fig. 7,

the highest and lowest lines were found to be for stripped reservoirs, while the line representing the Sudbury reservoir lay fairly in the middle. In other words, the stripped reservoirs had a little greater range in organisms than the unstripped, although there was no substantial difference in the average. Similarly, in the records for the reservoirs more than twenty years old in Fig. 8, which included the three oldest Boston reservoirs which were stripped and the Central Park reservoir of the New York system, the difference between the stripped and the unstripped was not significant.

There are two matters that are not taken into account in the above comparisons and which require to be mentioned. The plottings show the gross number of organisms of all kinds with only the qualification that they are rated according to size by the standard method that is used. Different kinds of organisms produce different tastes and odors, and in different amounts, and the gross numbers used in this case indicate only the general characters of the reservoir water, and some deviations are to be anticipated from the general results.

For a part of the period covered by these records copper sulphate has been used in the New York reservoirs from time to time where the growths were strongest, to the extent on an average of about one-third of a pound per million gallons of total output. This has no doubt tended to reduce somewhat the growths of organisms in the New York reservoirs.

A study of the records of the water for the nine years from 1906 to 1914 under the actual conditions of service gives no indication of any substantial advantage from the standpoint of organisms in the water that can be attributed to the stripping of those reservoirs which have been stripped. The results, therefore, fully confirm those of the earlier study and support the position of the New York Board of Water Supply in not stripping the Ashokan and Kensico reservoirs.

An Unusual Dam

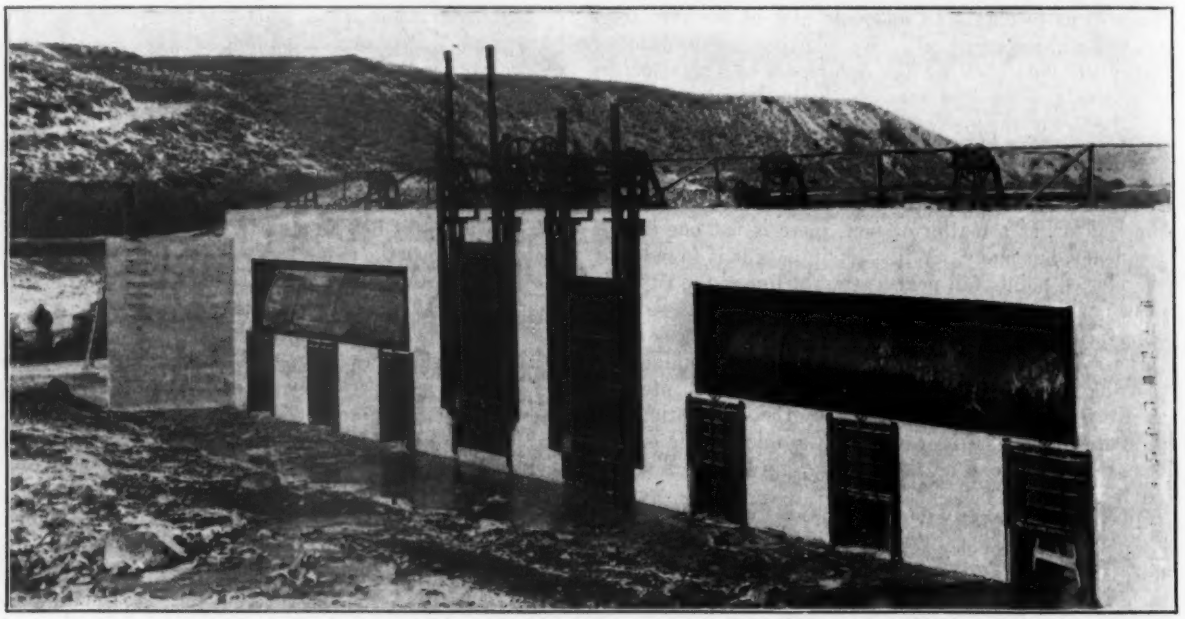
By Sylvester Q. Cannon*

A small impounding and diversion dam and measuring weir has been built recently in the Jordan River at Jordan Narrows, about 20 miles from Salt Lake City. The cost was borne jointly by four canal companies, the Utah Power & Light Co. and Salt Lake City.

The dam is of the hollow reinforced-concrete type, pierced with ten openings controlled by cast-iron sluice and steel radial gates. (See view and note.)

By closing the radial gates water may be impounded. The regulation of the river discharge during the irrigation season will be mainly by means of the devices in the two central openings.

These unusual regulating devices were installed under the direct instruction of the court, in order to permit the discharge of water either under pressure, by lowering the upper portion of the gate to form an orifice, or under free discharge, by raising the lower section to form a weir.



Upstream Face of Jordan Narrows Dam and Measuring Weir

(Length between wing walls, 90 ft.; height from floor to top, 18 ft., and from floor to high-water level, 14 ft. The six smaller openings in the base of the dam, three at either end, are each controlled by a cast-iron sluice gate. The two larger central openings, controlled by three cast-iron gates in each, serve as regulating devices, as do also the two 24-ft. rectangular adjustable weirs, which may be closed by four radial steel gates.)

Since three of the parties interested divert their share of the water just above the dam and the other three have their points of diversion some distance below, the intention has been to provide openings that would distribute the water as equitably as possible under varying conditions of flow. These gates will, however, require practically as much regulation as a simpler design, if not more.

The cost of the materials at the dam site was as follows: Sand, $1.43 per cu.yd.; gravel, $1.63 per cu.yd.; cement, $2.54 per bbl.; lumber, $30 per M. The cost of the reinforced concrete in place, including formwork and removing forms, was $11.41 per cu.yd. This cost was relatively high because of the many openings and the consequent difficulty of formwork. The dam was built under the joint supervision of the Court Commissioner, J. Fowson Smith, and the Engineering Department of Salt Lake City.

*City Engineer, Salt Lake City, Utah.

Motor Vehicles in Water-Works Service at Los Angeles, Calif.

By Burt A. Heinly*

SYNOPSIS—The Los Angeles Water Department, under Wm. Mulholland, Chief Engineer, has gradually changed from horse-drawn to motor vehicles for all classes of service. The equipment is described, operating costs are given, and some comparisons between horse and motor service are made.

After a decade of experiment and use of the motor vehicle, the Los Angeles City Water Department disposed of the remainder of its horses and wagons on Apr. 1, 1915. Ten years earlier, before trucks were to be had on the Pacific Coast, the department bought a small runabout from a local factory and made it over for light hauling. Two more were bought and rebuilt in 1908. A cost account was kept for 1909, and the three machines showed a saving of $92 per month each, as against horses, wagons and men doing equivalent work. Meanwhile, automobiles were bought for administrative purposes, and subsequently both trucks and cars of various kinds were added, until at the present time 38 machines of all sorts (Table I) are in use.

Purchase and Equipment—Purchase of all machines is made under the competitive-bidding system under broad general specifications for each class of vehicle. One unwritten requirement is that the bidder shall have in the city a model of the machine on which he submits his offer and that he shall here maintain a full assortment of repair parts. In most cases the specifications require that the factory rating and the cost of principal parts shall be included in the proposal.

Advertisement for bids on purchases under $500 is not required by law, but supplies of every nature excepting exclusive-agency articles must be purchased upon competitive bidding. Gasoline is purchased on annual contract. Until July 1, 1914, it cost 13½c. per gal.; since then the contract price has been 12c. per gal. Lubricant of the best manufacture is purchased at 38½c. per gal., in 50-bbl. lots. The department procures wholesale prices on all articles excepting special parts. Before making a purchase a requisition must be delivered to the seller; otherwise payment is refused. Trucks of less than 1½ tons are fitted with pneumatic tires; trucks of 1½ tons, with single solid, and over 1½ tons, with dual solid tires.

Maintenance—All repairs are made in the machine shop of the water department. Repair, service and leak cars are either discarded runabouts or new vehicles on which bodies have been built. These bodies, as well as those for new trucks, are built by the department at a saving of about 40% of the factory price.

The mechanical force is on duty from 7 a.m. to 4 p.m. It is thus made possible to inspect each vehicle before it leaves the yards—the cars must be on the work at 8 o'clock. One is kept as an extra and replaces any car that is sent to the shop for repairs. On an average, a car receives an overhauling once in 12 to 15 months.

Each foreman and every executive, with the exception of the chief engineer, drives his own car. The chauffeur for the latter is supposed to keep his car in condition, with shop work on it, when he is not driving. His hours are from 7:30 a.m. until 6 p.m., with Sunday and holiday work as well. He receives a salary of $80 per month. Drivers of trucks hauling small pipe and fittings do their own unloading. On heavy pieces they are given a helper.

Each car is given a number. All material and labor expended upon it in the shop is charged against it on a form which is filed with the foreman—a separate form for each vehicle repaired each day.

Every car is provided with a speedometer—that which comes with the car—and these have been read once a

*Secretary to William Mulholland, 645 South Olive St., Los Angeles, Calif.

TABLE I. DESCRIPTION AND USES OF MOTOR-VEHICLE EQUIPMENT, LOS ANGELES WATER-WORKS

Car No.	Make	Model	A.L.A.M. hp.	Weight, lb.	Capacity	Date Bought	Cost	Class	Use
5	Franklin	1907	15.6	1900	5-passenger	Oct. 12, 1907	$1900	Work car	Extra
7	Mitchell	1909	25.6	2550	2-passenger	Dec. 15, 1908	1150	Runabout	Meter department
9	Regal	1909	25.6	2300	2-passenger	July 17, 1909	1350	Runabout	Asst. supt.
10	Stoddard-Dayton	1908	27.2	3240	¼-ton	Feb. 23, 1910	600	Work car	Gate inspection
11	Autocar	1910	18	4300	1½-ton	Sept. 12, 1910	2350	Work car	Leak repairs
12	Autocar	1911	18	4300	1½-ton	May 25, 1911	2350	Work car	Leak repairs
13	Autocar	1911	18	4300	1½-ton	May 25, 1911	2350	Work car	Meter department*
14	Ford	1910	22.5	1450	2-passenger	June 21, 1911	500	Runabout	Mach. shop
15	Ford	1911	22.5	1450	2-passenger	July 1, 1911	750	Runabout	Office
16	Autocar	1911	18	4300	1½-ton	July 18, 1911	2350	Work car	Meter department
17	Autocar	1912	18	4300	1½-ton	Aug. 29, 1911	2350	Work car	Extra
18	Packard	1912	32.4	8175	3-ton	Nov. 15, 1911	3600	Truck	Pipe construction
19	Kissel Kar	1912	28.9	3650	½-ton	April 9, 1912	1450	Work car	Meter department
20	Moore	1912	32.4	7350	2-ton	April 9, 1912	2670	Work car	Meter department
21	Autocar	1913	18	4300	1½-ton	June 25, 1912	2300	Work car	Extra
22	Autocar	1913	18	4300	1½-ton	June 25, 1912	2300	Work car	Meter department
23	Kissel Kar	1912	28.9	3650	½-ton	June 25, 1912	1650	Work car	Meter department
24	Mack	1913	48.4	9100	3-ton	June 25, 1912	3250	Truck	Pipe construction
25	Kissel Kar	1913	28.9	3150	5-passenger	Oct. 20, 1912	1825	Passenger	Superintendent
26	Kissel Kar	1913	28.9	3150	5-passenger	Oct. 28, 1912	1825	Passenger	Chief Mech. Engr.
27	Autocar	1913	18	4300	1½-ton	Feb. 4, 1913	2150	Work car	Pipe construction*
28	Mitchell	1910	28.9	3150	½-ton	June 1, 1913	700	Work car	Gate repairs
29	Moreland	1913	27.2	6160	2-ton	June 30, 1913	2343	Truck	Pipe construction
30	Kissel Kar	1913	32.4	6225	2-ton	Sept. 23, 1913	2325	Work car	Meter department
31	Ford	1913	22.5	1475	2-passenger	Oct. 7, 1913	575	Runabout	Time keeper
32	Ford	1913	22.5	1475	2-passenger	Oct. 7, 1913	575	Runabout	Asst. Engr.
33	Pierce Arrow	1912	38.4	3770	4-passenger	Oct. 16, 1913	2500	Passenger	Public Service Com.
34	Pope Hartford	1913	36.1	4650	5-passenger	Oct. 16, 1913	1400	Passenger	Asst. Engr.
35	Mitchell	1910	28.9	2725	3-passenger			Runabout	Asst. Engr.
36	Franklin	1911	38.4	3450	4-passenger			Passenger	Extra
37	Franklin	1912	38.4	3475	5-passenger			Passenger	Extra
38	Buick	1914	22.5	2155	2-passenger	Apr. 22, 1914	1060	Runabout	Asst. Engr.
39	Moore	1914	32.4	7335	2-ton	Apr. 24, 1914	2351	Truck	Pipe construction
40	Ford	1914	22.5	1475	2-passenger	July 27, 1914	575	Runabout	Meter department
41	Ford	1914	22.5	1475	2-passenger	July 27, 1914	575	Runabout	Meter department

* In San Pedro section.

TABLE II. OPERATION AND MAINTENANCE COST OF MOTOR-VEHICLE EQUIPMENT, LOS ANGELES WATER-WORKS, IN 1914

Car No.	Make	Costs: Gas	Oil	Material	Labor	Tires	Total	Mileage	Cost per Mi.
Repair and Service Motors									
16	Autocar	$135.79	$23.80	$218.93	$106.30	$223.33	$708.24	7,169	$0.098
19	Kissel	203.49	50.59	139.94	75.54	291.49	761.05	12,798	0.06
20	Moore	149.31	40.44	149.40	198.76	466.71	1004.62	5,124	0.19
22	Autocar	162.91	43.68	41.14	29.68	150.84	428.25	6,814	0.062
23	Kissel	221.07	63.01	256.23	134.57	259.83	934.71	12,077	0.077
30	Kissel	157.14	38.95	54.89	35.46	.....	286.44	5,886	0.049
40	Ford	32.55	6.64	12.75	1.77	5.72	59.43	5,550	0.01*
41	Ford	44.82	11.36	9.22	6.74	19.80	91.94	6,852	0.013*
Total		$1107.08	$278.67	$882.50	$589.01	$1416.72	$3374.68	62,270	
Av. cost per car		138.38	34.83	110.31	73.62	177.09	421.83	7,783	0.07
Av. cost per day		0.46	0.116	0.367	0.245	0.59	1.40	26	
San Pedro Service									
13	Autocar	$121.88	$22.67	$96.15	$51.82	$62.47	$354.99	7,481	$0.047
27	Autocar	166.73	35.37	43.10	60.10	.....	305.30	9,627	0.031
Total		$288.61	$58.04	$139.25	$111.92	$62.47	$660.29	17,108	
Av. cost per car		144.30	29.02	69.62	55.96	31.24	330.14	8,554	0.039
Av. cost per day		0.48	0.09	0.23	0.19	0.10	1.10	28	
Aqueduct Service									
34	Pope Hartford	$394.51	$125.40	$270.47	$143.81	$721.89	$1656.08	15,293	$0.108
38	Buick	76.90	11.00	74.53	32.95	150.66	346.04	5,868	0.059†
Total		$471.41	$136.40	$345.00	$176.76	$872.55	$2002.12	21,161	
Av. cost per car		235.70	68.20	172.50	88.38	436.28	1001.06	10,580	0.084
Av. cost per day		0.78	0.13	0.57	0.30	1.44	3.33	35	
Extra Cars									
36	Franklin	$140.47	$24.82	$161.80	$159.37	$45.79	$532.25	9,455	$0.056
37	Franklin	104.25	13.54	207.95	129.62	277.82	733.18	9,624	0.076
Total		$244.72	$38.36	$369.75	$288.99	$323.61	$1265.43	19,079	
Av. cost per car		122.36	19.18	184.87	144.50	161.80	632.71	9,540	0.066
Av. cost per day		0.41	0.06	0.61	0.48	0.54	2.10	31	
Administration									
7	Mitchell	$61.78	$8.84	$144.44	$106.14	$41.08	$362.28	2,482	$0.146
9	Regal	196.90	47.82	401.83	222.61	91.17	960.33	10,740	0.09
15	Ford	77.06	11.60	133.02	113.90	61.33	396.91	8,139	0.048
25	Kissel	174.52	43.18	101.34	129.84	149.29	598.17	13,259	0.045
26	Kissel	156.62	37.77	99.04	184.79	328.58	806.80	18,130	0.044
31	Ford	109.50	10.38	73.62	49.64	69.42	312.56	13,148	0.023
32	Ford	129.76	37.78	189.14	151.24	146.38	654.30	19,608	0.033
33	Pierce	127.85	9.92	72.17	94.87	271.88	576.69	7,152	0.08
35	Mitchell	133.77	32.90	202.92	223.83	41.77	645.22	9,800	0.066
Total		$1162.76	$240.19	$1399.52	$1276.82	$1200.90	$5303.23	102,458	
Av. cost per car		129.19	26.69	155.50	141.87	133.43	589.24	11,384	0.063
Av. cost per day		0.43	0.09	0.51	0.47	0.44	1.99	38	
Light and Heavy Trucks									
10	Stoddard-Dayton	$165.08	$33.96	$442.89	$161.99	$152.66	$956.58	9,990	$0.095
11	Autocar	215.15	41.38	165.40	90.38	181.51	693.82	12,410	0.056
12	Autocar	216.13	37.80	244.31	122.55	324.48	945.27	10,094	0.093
14	Ford	16.90	1.29	77.91	66.67	49.26	212.03	1,792	0.118‡
17	Autocar	157.53	37.74	171.21	128.59	164.62	659.69	8,957	0.073
18	Packard	329.46	84.66	117.78	73.38	452.99	1058.27	13,613	0.08
21	Autocar	152.97	39.00	27.97	39.37	152.54	411.85	7,740	0.053
24	Mack	443.93	100.18	98.85	90.50	378.87	1112.33	11,112	0.10
28	Mitchell	123.11	44.75	194.89	106.66	33.93	503.34	8,762	0.057
29	Moreland	192.30	35.99	35.01	60.14	48.33	371.77	8,127	0.045
39	Moore	147.27	26.68	48.10	44.17	27.75	293.97	5,647	0.052‡‡
Total		$2159.83	$483.43	$1624.32	$984.42	$1966.94	$7219.44	98,244	
Av. cost per car		196.44	43.95	147.68	89.50	178.81	656.40	8,931	0.074
Av. cost per day		0.65	0.15	0.49	0.30	0.59	2.18	30	
Recapitulation									
Total for 34 cars		$5439.41	$1234.89	$4778.34	$3427.84	$5843.42	$20,682.90	309,320	
Av. cost per car		159.98	36.32	140.54	100.82	171.90	608.32	9,098	
Av. cost per day		0.53	0.12	0.46	0.33	0.64	2.03	30	

* Six months' service. † Nine months' service. ‡ Four months in Owens Valley. ‡‡ Eight months' service.

month. This has not been satisfactory and henceforth each driver will be furnished with a book of tickets and required to turn in a daily reading. Also, when a new tire is to be placed the mechanic will record the wheel location and the speedometer reading, so that a tire mileage will form a part of the record.

OPERATING COSTS—Table II, it must be admitted, does not give the full information required for a final judgment as to the operating value of the different cars. Re-tiring or a thorough overhauling within a twelve-month, or both, run up mileage costs for the period to an unfair degree. At least a three years' record seems necessary and this the department is undertaking, with closer attention and carefulness than heretofore.

Allowing 300 work days in the year, the average daily mileage on all cars, excepting administration, ranges from 25 to 30, and the cost per mile is around six and seven cents, with the exception of the San Pedro and Aqueduct field cars. The former are two Autocar trucks used for general-utility work on an independent water system, located 18 mi. from headquarters. The Aqueduct field touring cars run high in operation because they are used under the onerous conditions of mountain and desert.

The apparently low daily mileage for the 34 vehicles is explained by the fact that for the larger part of the day, in water-works operation, the cars are inoperative. For example, in pipe construction they reach the place of work and as a rule are little used until the time for the gang's return. Here their great value is to be found in their "readiness for service"—the rapidity with which small material and fittings which exigency demands can be brought from the shop or in enabling the foreman to keep constantly in touch with two or three small gangs working in the same vicinity.

It will be noted that no allowance is made for depreciation. It is too well known for comment that the life of a motor vehicle depends upon its driver, and in the department there is a wide divergence of opinion as to what

comprises a proper depreciation charge. Mr. Read, in his charges against his department, makes an allowance on service and leak wagons of $120 per year for this purpose. This is depreciation on his light cars of over 20% per annum. Mr. Brooks sees no reason why, with the present perfection of automobile building, an honestly built machine should not last for ten years and even then have some salvage value.

In trucking, large pipe is hauled under contract, the contractor doing all his hauling by teams. Only the stock pipe (4- to 10-in.), fire hydrants, gates, and the like, are handled by department trucks. Conditions often require less than full loads and also diminishing loads over considerable distances. Therefore, it is not practicable to give a ton-mile cost.

HORSES VS. AUTOMOBILES—During the past summer a comparative study was made of the cost of horse-trucking under contract and automobile trucking by the department. The pipe was hauled over the same course of six miles. The contractor used a four-horse truck loaded to seven tons and was able to make only one round trip of a total of 12 mi. in a 9-hr. day, the department furnishing one man in the unloading. At the contract rates, the cost of haul was $1.20, exclusive of labor for unloading. The department used the three-ton Packard loaded to 3½ tons and the two-ton Mack loaded to 2½ tons. Each truck made four round trips in an 8-hr. day, the expense being as follows: Two drivers at $3.50 each; one helper at loading, $2.50; gasoline, $2.40; lubricant, $0.71; tires and upkeep, $3.31; total, $15.92, or a cost of $0.667 per ton. Figuring the cost of the contractor's helper, each department automobile, although of light tonnage, hauled nearly twice as much pipe in a shorter time at a cost of less than half the contractor's charge.

From actual records Mr. Read, meter superintendent, estimates that one of his light motor trucks does as much work as two two-horse wagons. Moreover, a team limits the gang, whereas the truck makes the gang limit itself on the actual work which it is able to do.

The largest number of horses at the water-works yards was 43, for which four stable men were required—one at $2.75 per day and three at $2.50. With hay at $16 per ton and barley at $30.60, the average itemized expense per head per month was: Hay, $8; barley and bran, $3; shoeing, $2; labor, $7.15; or a total of $20.15 per month, with no allowance for depreciation or doctor's bills. Upkeep of wagons and harness averaged hardly less than $1 per month.

If we accept Mr. Read's ratio of one machine replacing two two-horse wagons, the department at the present time would require, at the very least, 100 head of stock. On the above monthly charge for horse use this would amount to an operating charge per year of $24,780, as against $20,682 paid out last year for motor-truck service.

The purchase prices of machines are shown by the table to range from $575 to $3600. Horses have cost the department from $150 to $225 per head; wagons, from $75 to $150 and $175; harness, from $15 to $30. A two-horse rig at the present time costs nearly or quite as much as the cheapest service auto. The average life of a horse on city streets is from six to seven years, after which he becomes practically useless for department purposes.

The motor vehicle has made possible the continued centralization at its water-works yards of the whole activities of its construction forces. In a decade the municipality has grown in population from 219,000 to 550,000 and in area from 60 to 107 sq.mi. In the same interval the pipe mileage has increased from 358 to 629; fire hydrants, from 861 to 4319; meters, from 3400 to 63,000; and services, from 39,958 to 84,000.

Without motor vehicles the maintenance and operation of the works would have compelled the partition of the city into five districts, with consequent purchases of land, erection of buildings, duplication of records and supplies and increase of clerical forces; and also with a loss of efficiency from the inability of executive heads to keep in close touch with every man on the job. With automobiles, the assistant superintendent visits his gangs from one to three times a day, the chief mechanical engineer makes a daily inspection of the eight pumping stations, with an average journey of 60 mi. per day, while the large works of aqueduct distribution, 24 mi. to the northwest of the city, are made readily accessible to the chief engineer and his assistants.

A Unique Water-Supply

The water-supply of Newark, N. J., is unique in that the city now owns in fee simple some 62% of the 63 sq.mi. of the watersheds from which the supply is drawn, and in the course of a few years will probably own all of these watersheds. This case is hardly parallel to that of Seattle and other far western cities which own all or large parts of the watersheds of their municipal water-supplies, for the impounding reservoirs of those cities are in comparatively unpopulated wildernesses, while Newark's reservoirs are within a 30-mi. radius of New York City and hence in one of the most densely populated parts of the United States.

Newark is, however, exceptionally fortunate in having its reservoirs located in one of the few sections in the vicinity of New York City which never has had a large population or particularly high land values. Owing to the farsightedness of the officials in charge of the City's Department of Public Works, the policy was long ago adopted of acquiring the watershed lands by purchasing from time to time whatever parts of them were offered for sale at reasonable prices; thus the city has obtained its land at fair prices, has prevented real-estate speculation and has kept that part of the state from becoming a suburb of the metropolitan district and a menace to its water-supply.

FIG. 1. BROWN'S HOTEL, NEWFOUNDLAND, N. J., PURCHASED AND DEMOLISHED BY THE CITY OF NEWARK

In 1889 the City of Newark entered into a contract

with the East Jersey Water Co. for the building and operation of water-works to furnish an ultimate daily supply of 50,000,000 gal., with the option of taking over the works in 1900, which was done. The impounding reservoirs are now five in number and are located on the Pequannock River and its tributaries, some 30 mi. northwest of the City of Newark, in the northern part of New Jersey. The lower reservoir, known as the Macopin intake, is connected with the city by two parallel steel mains, one 42 in. and the other 48 in. in diameter. There are three distributing reservoirs nearer the city.

FIG. 2. VILLAGE OF MACOPIN, BUILT BY THE CITY OF NEWARK FOR EMPLOYEES ON ITS WATERSHED

The policy of purchasing the watershed lands was begun by the Water Board upon the recommendation of Morris R. Sherrerd as chief engineer of the water-works in 1896, and has been followed continuously and systematically ever since. The water-supply has been enlarged and two new reservoirs added under the city ownership, yet the whole sum expended in the acquisition of 39 sq.mi. of watershed lands has not exceeded $1,100,000 to date, and it is probable that the whole watershed of 63 sq.mi. will be obtained for much less than $2,000,000. Single purchases have ranged all the way from 0.14 acre to 2411 acres, and from $300 to $170,000. Coincident with the purchase of land and the removal of habitations there has been a reduction of the population from 2000 to 900.

FIG. 3. MACOPIN RESERVOIR; EVERGREEN HEDGE TO SCREEN OFF DUST AND LEAVES

One of the most interesting of the land purchases was by an unusual contract with the East Jersey Water Co. during 1900 to 1902, by which surplus water then going over the spillway at the city intake was given to the water company in exchange for 3900 acres of land in the watershed owned by the water company. The terms of this contract were 2,000,000 gal. of water for one acre of land, which was an advantageous bargain from both points of view, since the water company sold the water to consumers in Jersey City and Bayonne at a large profit and the city acquired a block of the watershed lands for something which would otherwise have been thrown away. Another interesting purchase was that of Brown's Hotel (Fig. 1) at Newfoundland, once a famous country hostelry and a stopping place for the engineers who built the dams. This was one of the most valuable pieces of property in the watershed and, with 800 acres of land, cost the city $170,000.

The material and furnishings of the buildings torn down are utilized for the construction of houses for the watershed employees, of whom there are 80. These houses are outside of the watershed in a little water-works village called Macopin (Fig. 2). Here are nine first-class houses of 6 or 7 rooms, with bath and running water, which are let to employees for $7 per month.

The village will eventually include a school and possibly other public buildings. Its sewage is treated at a small disposal plant before discharging into the Pequannock River below the water-works intake.

Most of the employees of the watershed are laborers at $1.75 per day; many of them have been employed there since the days of the water company. These laborers are chiefly occupied in forestry work, in cleaning up the watershed and improving roads. Eight teams are required to take the laborers to and from their work.

There are only two inspectors, whose sole duty it is to enforce the rules and regulations of the watershed area. One makes his tours by motor cycle, the other with a horse and wagon. On Saturdays, Sundays and holidays three men are detailed to motor-boat patrol duty on the reservoirs to enforce rules against fishing, which is now prohibited because of the practical impossibility of enforcing sanitary precautions among the hosts of fishermen who frequented the reservoirs and brooks.

The forestry is in an elementary stage, but during the last five years over 1,000,000 trees have been set out. About 350 acres have been planted and this year some 200 acres will be added. Much of the watershed is already well wooded, but the recent chestnut-tree blight and the ice storm of last winter destroyed many trees and kept the workmen busy trimming out dead wood. Eventually, however, it is believed that a municipal forest will be created which will return a considerable income upon a not very large investment. The planting so far has been chief-

ly of Scotch pine, ash, locust, Norway pine and black walnut, the last being native to the watershed area. The development and maintenance of the reservoirs and watersheds are under the immediate supervision of Hermann Rosentreter, Principal Assistant Engineer, Department of Water, Newark.

A 90-In. Circular Gate for a 120-Ft. Head

An interesting design of a circular sluice gate (shown in the accompanying views) has recently been developed by the Coldwell-Wilcox Co., of Newburgh, N. Y., for a new 90-in. sewer in Sacramento, Calif. This gate is in a gatehouse on the outlet end of a 90-in. reinforced-concrete discharge pipe from a new pumping station. This forms part of a combined sewerage system being constructed to care for about 6020 acres of territory now annexed to the city. During low water in the Sacramento River the discharge line empties by gravity into the river, but at high water the pumps work against a maximum head of about 30 ft.

The appended extracts from the specifications under which the gate was designed explain the high head assumed, which is far beyond the static conditions of service:

It must be of an approved and standard type, having cast-iron rings and disk heavily ribbed and be bronze mounted throughout, having accurately machined, renewable babbitt disk seats.

The valve must also be provided with handwheel for hand operation. It must be capable of safely standing a hydrostatic pressure of 50 lb. per sq.in. as well as closing safely at any rate up to five minutes from clear open to closed position should the flow through the discharge pipe be reversed and a velocity of 30 ft. per sec. be obtained during a temporary interruption to the electric service.

The hydraulic cylinder must be so proportioned that it can operate this gate under the above-cited conditions when the cylinders are operating with water service corresponding to a head of 260 ft.

It must be designed to withstand safely an unbalanced head of 50 ft.

The frame for sluice gate shall be an integral casting of a combination flanged and thimble collar type, to be standard to provide for bolting to the masonry. The thimble shall be 6 in. in length and of sufficient strength to carry the lifting stresses for lifting the gate when it is wedged in closed position.

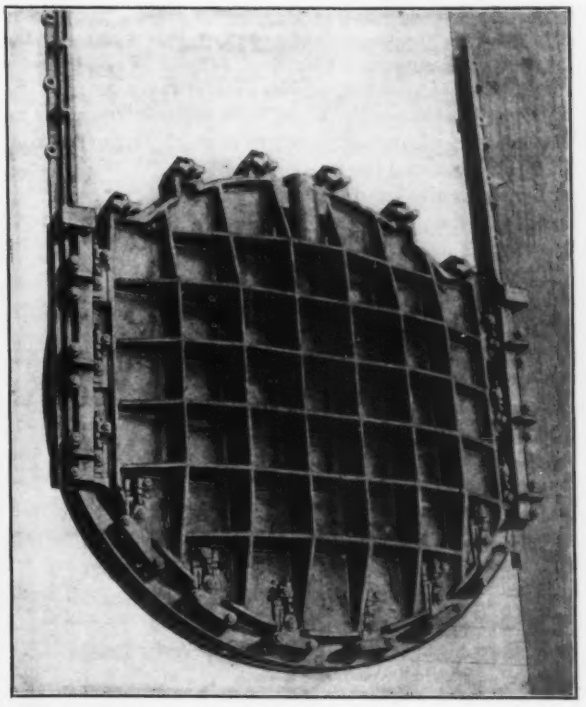


FIG. 1. THE 90-IN. SEWER SLUICE GATE AT SACRAMENTO

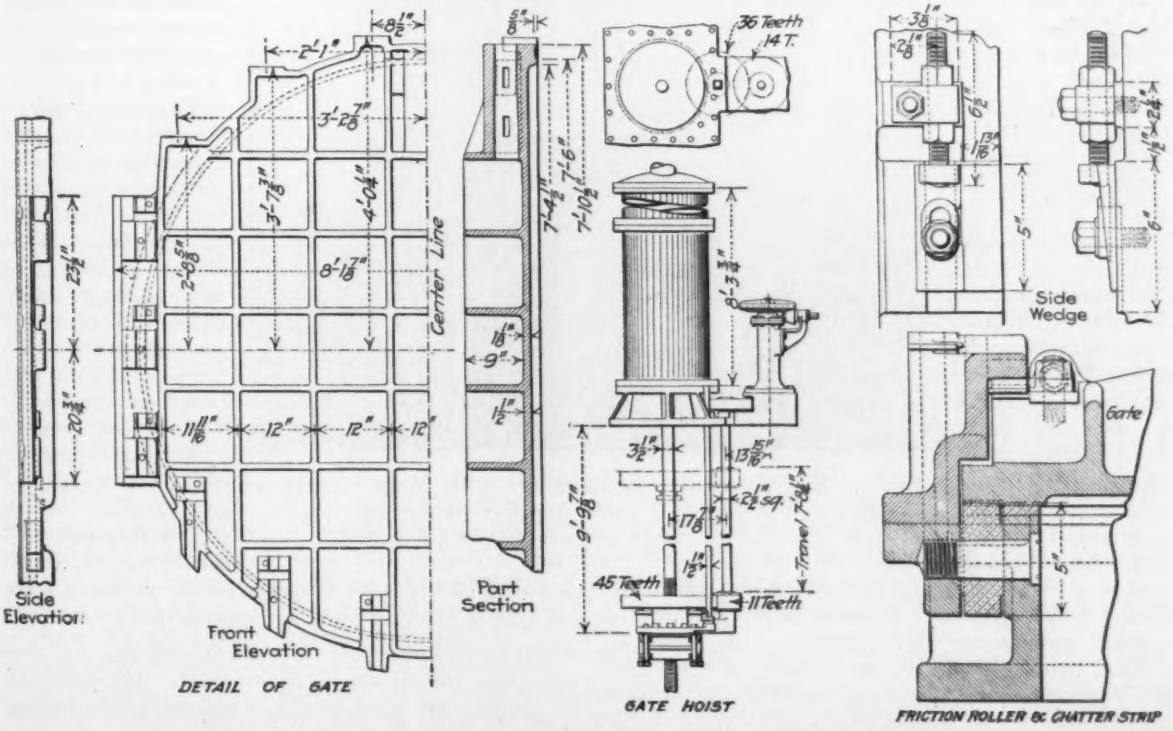


FIG. 2. DETAILS OF 90-IN. SLUICE GATE AND HOIST FOR 120-FT. HEAD

The Structural Features of a 435-Ft. Tower, Panama-Pacific Exposition

By Frank S. M. Harris*

The main-entrance tower at the Panama-Pacific International Exposition, San Francisco, Calif., occupies a commanding position at the head of the Court of the Universe. Rising to a height of 435 ft., it overtops the surrounding domes and arches by 250 ft. or more. The structure is used wholly for esthetic purposes and plays no part in the utilitarian requirements of the exposition. It might be termed the principal architectural accent of the exposition.

The tower is distinguished not only by its height—probably the greatest of any structure west of Chicago except the new Smith Building in Seattle—but also by the great clear height of the barrel vault which pierces it from north to south. This vault is 110 ft. high and 60 ft. wide; it is spanned by steel arches 115 ft. in span and 126 ft. high from bottom hinges to center of lower chord. Immediately over the vault rests the main mass of the huge structure, supported by the steel arches.

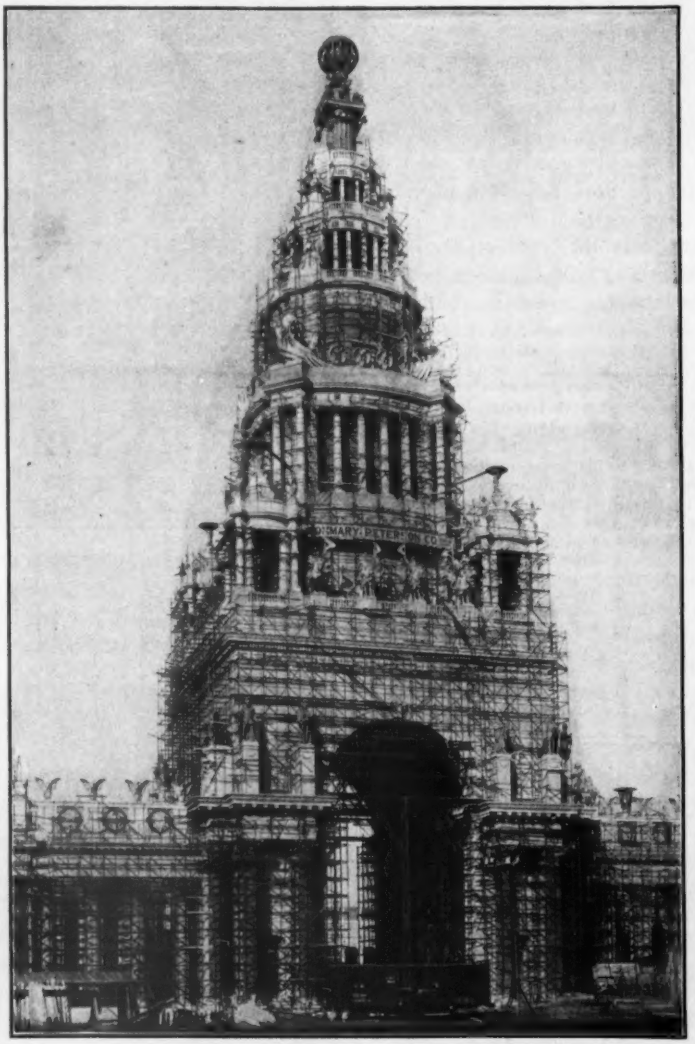


Fig. 1. The Tower of Jewels, the 435-Ft. Main Entrance Tower of the Panama-Pacific Exposition, San Francisco

The "Tower of Jewels"

The novel and spectacular feature of this structure results from the many imitation jewels hung upon its surface in such arrangement as to pick out its various parts. These jewels, which are from 21 to 47 mm. in diameter, have a cut similar to the Sumatra cut, the exact number of facets, however, having been computed and checked by experiments. These imitation gems are of Austrian manufacture of the highest grade, with tin-polished facets. About 100,000 of them were required in the tower scheme, and they are mounted in such a manner as to have easy play in the air currents, which causes them to glisten and sparkle when subjected to the searchlight beams during the night illumination.

General Structure of the Tower

To a height of about 150 ft. above ground, the walls of the tower rise vertically, forming a huge block 120 ft. square in plan. The sides of this block are pierced in the N-S direction by a barrel arch, whose soffit is 119 ft. above the court. Above, the structure consists mainly of a series of cylindrical portions of variable and rapidly decreasing diameter, composed of balustrades, colonnades and heavy belt-courses. The outstanding ornament upon these upper decks is elaborate.

Flanking the tower, east and west, and extending the whole width of the Court of Honor, are colonnades surrounding rectangular courts which contain the Fountains of Youth.

The main frame of the tower is of steel, supporting an exterior shell of timber, plaster and staff. The great rectangular box, below El. 152 ft., as well as the flanking colonnades, are self-supporting timber structures, but slightly tied back to the steel frame. The coffered ceiling of the barrel vault is suspended from the steel arches. Above the rectangular section, the successive decks are carried upon timber brackets, resting in seats provided on the steel frame and secured to it by rods and clip angles. Upon these decks are framed the walls and outstanding ornaments of the tier above. The details of the framing are given in Fig. 2 and the accompanying views show the structure during erection.

*Assistant Structural Engineer, Panama-Pacific International Exposition.

Steel Frame

The steel structure consists mainly of six frames, parallel to each other in plan, two of them vertical, the other four battered. The two vertical frames (interior frames) rise to El. 364, being 20 ft. c. to c. throughout. The two exterior frames are battered so as to meet the vertical interior frames at El. 335. The two intermediate frames also batter, at half the rate of the exterior frames, but are discontinued at El. 227 to simplify the construction. Above El. 335 all columns are vertical. Below El. 152 the six frames are constructed as two-hinged arches with elastic ties, their feet resting on twelve cast-steel spherical bases on reinforced-concrete slab footings capping pile clusters.

Cross-bracing is introduced between the six frames, both in the planes of the main batter posts and in the planes of the lower vertical members of the main arches, from El. 126 to the ground. Bracing is further provided in the planes of the kneebraces and the hip members of the arches. These diagonal tension members take the form of square upset rods, except in the immediate vicinity of the portals on the E-W axis and adjacent to the elevator shaft in the northeast quadrant, where stiff diagonals composed of four angles and lacing are employed to take either tension or compression that may come upon them due to the discontinuity of the tension diagonals.

Horizontal bracing is placed at 20-ft. intervals in the lower legs of the frame, and at approximately 25-ft. intervals (immediately below the floors) throughout the upper reaches of the frame.

Dead-Loads

The exterior walls consist only of staff and plaster on metal lath spiked to the studding and furring timber. With the exception of a loose plank floor at El. 152, intended to protect the main plaster arch from injury by falling objects, there are no complete floors in the building, the several decks extending but a few feet inside the exterior walls.

Inasmuch as spectators are not to be permitted within the tower, even these floor systems were considerably lightened in the design. A live-load of 50 lb. per sq.ft. was taken to insure stiffness, but was not carried into the column investigations. There are no interior partitions. The staff was taken at 25 lb. per sq.ft., including the timber backing sticks, but not the main studding of the walls. Plaster on metal lath was taken as 20 lb. per sq.ft. The weights were thus quite definitely known.

Wind Loads

Wind load of 30 lb. per sq.ft. vertical projection was assumed upon all exterior surfaces—the pressure which is to be expected from a wind velocity of 100 mi. per hr. This high velocity was chosen in view of the isolated position of the tower as regards other high structures and the unobstructed sweep of winds from the Pacific Ocean. Full reduction for cylindrical surface by the application of a two-thirds factor was not made, it being thought that the outstanding ornament would form pockets which would retain the wind sufficiently to considerably offset the effect of the cylindrical surfaces in reducing pressure. A compromise was therefore made, in the use of an eight-tenths factor, making the wind load, as used, 24 lb. per sq.ft. vertical projection.

No wind was assumed on the steel frame below El. 152, as the timber structure, which is self-supporting, protects the steel frame from the horizontal wind loads.

Below El. 152, the main frames were considered to act as two-hinged arches, carrying the vertical and horizontal loads from the structure above as concentrations on their upper chords at the points of the column connections. However, as a precaution in the event that for any reason the frames should not act as two-hinged arches, the stresses were investigated considering that portion of the arch between El. 126 and 152 to act as a simply supported truss resting upon the lower legs of the arch. This assumption, as is to be expected, greatly increased the stresses in the upper and lower chords at the center of the span. While this condition of stress could never obtain in full degree it was thought advisable to limit the intensity of stress in the several members under these assumptions to 30,000 lb. per sq.in.

In the matter of temperature stresses, it is evident that the inclosing of the steel frame within the timber and plastered walls, and the burying of the tie members under the pavement of the court, materially reduce the temperature range to which the steel arches are subjected. For the two-hinged arch with an elastic tie, temperature stresses arise only from a difference in temperature between the tie and the arch proper. A difference of 40° F., if distributed equally above and below the closing temperature, causes a change in length in the tie member of only $\frac{3}{16}$ in. This is equivalent to the effect of an axial force of 56,300 lb., which is but 30% of the total stress in the tie member. It was, therefore, thought justifiable to neglect further consideration of the temperature stresses, particularly in view of their rare occurrence in this magnitude and also since there is a considerable factor of safety in the tie members offered by the lateral resistance of the foundation piers to spreading.

As to the transverse frames, it was assumed that the columns would take vertical reactions due to overturning moments in proportion to their distances out from the center or neutral axis of the frame. The total horizontal shears at the several levels were distributed over the columns equally, inasmuch as these columns take this shear not in proportion to their stiffness in bending, but as a result of the truss action of the diagonals.

The stresses resulting from a wind striking the tower at an angle of 45° to the axis, it was reasonable to suppose, might give a condition of maximum stress in the four corner columns, acting as they do simultaneously in the main and the transverse frames. For this purpose, it was again assumed that the columns took vertical wind stresses in proportion to their distance out from the neutral axis of the tower. Section moduli were computed for the different horizontal sections that were to be investigated, about an axis perpendicular to the wind—at 45° to the axis of the tower. Combining the vertical wind stresses, so computed, with the dead-loads in the columns, it was found that the stress was in every case from 5% to 25% lower than the maximum stress under wind parallel to the frames.

The working stresses used in the design were taken at 16,000 lb. per sq.in. for tension and $16{,}000 - 70\frac{l}{r}$ for compression. Where axial wind loads were combined with dead-loads, an eight-tenths factor was introduced into the former before combining with the dead-load, in

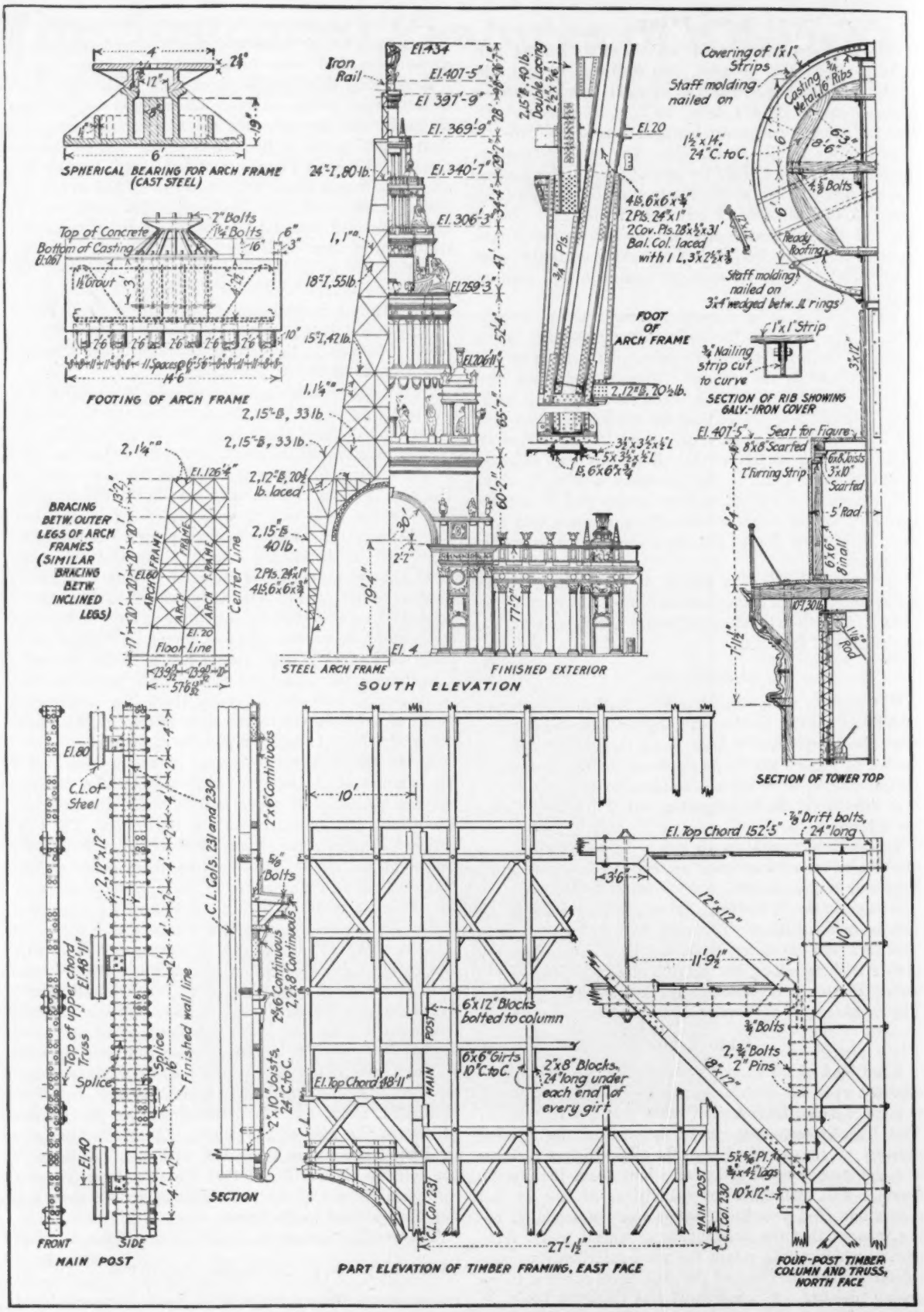

SPHERICAL BEARING FOR ARCH FRAME (CAST STEEL)
FOOTING OF ARCH FRAME
BRACING BETW. OUTER LEGS OF ARCH FRAMES (SIMILAR BRACING BETW. INCLINED LEGS)
STEEL ARCH FRAME
FINISHED EXTERIOR
SOUTH ELEVATION
FOOT OF ARCH FRAME
SECTION OF RIB SHOWING GALV.-IRON COVER
SECTION OF TOWER TOP
SECTION
FRONT
SIDE
MAIN POST
PART ELEVATION OF TIMBER FRAMING, EAST FACE
FOUR-POST TIMBER COLUMN AND TRUSS, NORTH FACE

FIG. 2. FRAMING OF THE TOWER OF JEWELS

effect working the steel at 20,000 lb. per sq.in. under the wind load. Diagonal tension rods were designed at 20,000 lb. per sq.in. To guard against any weakness due to faulty end connections between rods and gusset-plates, the pins and clip angles were given 20% additional strength. An initial tension of 5000 lb. was added to the computed stress of each rod.

For the structural details, shear was taken at 11,000 lb. in shop rivets, 8800 lb. in field rivets and turned bolts, and 7000 lb. per sq.in. on ordinary bolts, with bearing in all cases twice as great. Turned bolts were used in vertical splices in columns, ordinary bolts in all other positions.

Timber Framing

The main cube of the tower below El. 152, and the colonnades to the east and west of the tower proper, are self-supporting timber structures.

Along the east and west walls of the cube, a series of two-post columns extends from the foundations to El. 152. These are tied together by bracing trusses which, in addition to their stiffening function, support the studding for the exterior plaster walls. The north and south elevations of the cube are framed with four-post columns also tied together with bracing trusses. Above the main arch the walls and the floor at El. 152 are carried on heavy timber trusses of 64-ft. span, placed in pairs one above the other, and so tied together as to form a thoroughly stiff portal across the archway. From these trusses circular furring arches of timber were suspended for the support of the arch ceiling.

The colonnades are also self-supporting timber structures, the tops of the colonnade columns being built into horizontal trusses, which in turn are carried into the steel frame at the interior or tower end and into a large four-post abutment tower at the outboard end.

The timber framing above El. 152, though comparatively light, was considerably complicated by the batter of the steel frame and the diagonal bracing rods. The circular plan of the decks necessitated the scarfing of many of the large sticks.

In the timber design the following working values were used:

	Lb. per Sq.In.
Tension with grain	1,200
Compression, end bearing	1,600
Compression, cross-grain	350
Bending	1,500
Shearing with grain	150
Modulus of elasticity, dead-load	1,000,000
Modulus of elasticity, live-load	1,500,000
Columns 1200; over l/d = 15,	1,600 (1-l/60d)

Foundations

Considerable attention was given to the foundations, in view of the unknown stresses to which the two-hinged arches would be subjected were serious unequal settlement to take place.

Ten test borings were put down at symmetrically disposed points over the area of the tower. These resulted in the finding of a yellow hardpan at depths ranging from El —35 to El. —47, the ground surface being at El. +4. Immediately overlying the hardpan was a softer layer of sand and clay from 12 to 15 ft. thick. Superimposed upon this were successive strata of a wet blue clay and sand and a loose yellow sand, each from 10 to 12 ft. in depth.

In the driving of the piles, it was found practically impossible to get them down more than 10 to 12 ft. without serious brooming and splitting, the worst feature of which was the uncertainty as to whether or not the damage was serious, as telescoping was frequent. This depth was considered insufficient against the horizontal wind shears. Arrangements were made for the simultaneous driving and jetting of the piles, a thoroughly satisfactory solution of the difficulty, though even with the jet the penetration per blow under the 5-ton hammer was frequently less than 1 in. after the first 10 ft. of driving. After this first bringing up, the piles drove slowly and uniformly until the hardpan was reached, in which they were practically brought to refusal. Although designed for a 20-ton load, these piles as driven could scarcely be expected to settle under a load of twice this amount.

The specifications called for yellow pine or red Douglas fir (Oregon pine) piles, unpeeled, with a butt diameter of 14 in. clear of the bark, after cutoff, and a minimum diameter at the tip of 8 in. for piles not exceeding 50 ft. in length and 7 in. for lengths exceeding 50 ft. In all, some 600 piles were driven, which at an average length of 45 ft. gave 27,000 lin.ft. of driving.

The piles are in clusters under the feet of the 12 main columns, the numbers in the groups ranging from 25 to 42. The piles in each footing are capped by a steel-reinforced block of 1 : 6 concrete.

Six concrete troughs were placed for the support of the steel tie members between the pins of the main arches. These troughs were for the dual purposes of heat insulation and protection from bending under settlement of the roadway, the ties being placed as close to the surface as practicable. After the steel erection was complete, these troughs were filled with a 1 : 8 concrete.

With the exception of the large timber columns of the tower proper, all timber framing in the colonnades and the small courts is carried on spread footings, with sand-boxes introduced where necessary to offer anchorage against overturning wind moments.

Spherical Cast-Steel Bases

For the feet of the two-hinged arches, it was thought that some provision should be made to guard against indeterminate bending in the legs, such as might result from unequal settlement of the foundations, unsymmetrical wind or temperature loads, or other such causes. A uniform distribution of the load was, of course, desired between the cast-steel bases and the concrete of the footing. All of these ends were accomplished by the use of a segmental spherical bearing. Also such a bearing greatly facilitated the closing of the arches in erection.

These bases are of cast steel, circular in plan, and 6 ft. in diameter. A concave spherical bearing, turned to a radius of 2 ft., was supported, concave upward, upon 16 radial ribs. Into this socket was placed the convex bearing shoe which bolts to the structural steel of the leg. Anchor bolts, 2½ in. in diameter, embedded in the concrete of the piers, extend up through the upper and lower sections of the cast-steel base and through the structural steel of the main member, thereby insuring against any tendency of the ball portion of the bearing to leave its seat, as under excessive wind pressure or earthquake impact.

Steel Erection

For the erection of the two-hinged arches, the steel contractor constructed a timber traveler 50 ft. square and of height sufficient, with blocking, to reach the lower

chords of the arches. The traveler was erected in position for the setting of the two interior arches. The bottom of the traveler rested upon steel rollers traveling upon a timber track shod with ¼-in. plates. The lower timbers of the traveler were protected by short lengths of 12-in. channel, placed with flanges up.

The upper deck of the traveler projected over the face of the framework a couple of feet, permitting the erection of the steel arch immediately against the side of the timberwork. This overhang prevented interference in the setting of the kneebraces. On the upper deck of the traveler, a 20-ton stiffleg derrick was so placed as to pick up the material directly from the receiving and sorting yard to the south and west of the tower site.

The 12 first-tier columns, some 50 ft. in height, were first set, about the entire building, and guyed back to deadmen, with sufficient of the horizontal struts in place

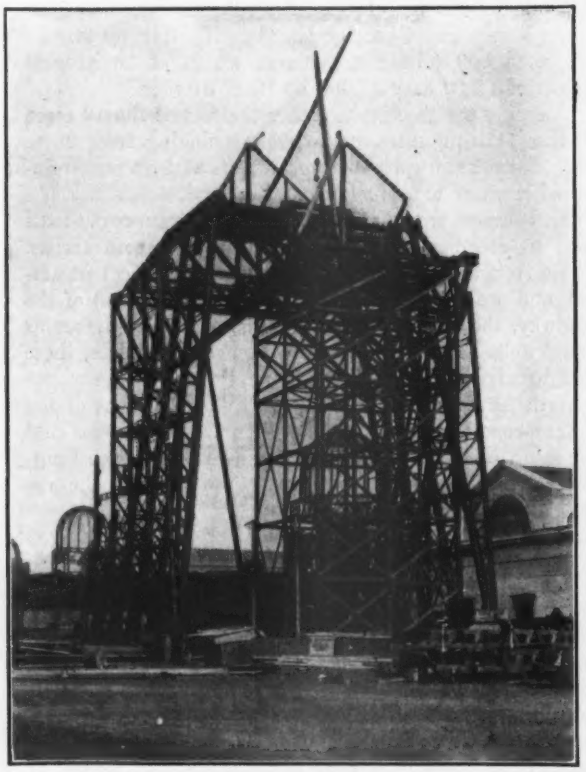


FIG. 3. ERECTION OF THE STEEL TWO-HINGED ARCHES OF THE TOWER OF JEWELS

to steady them laterally. The second-tier columns on the four central frames were then placed, after which the two interior frames were carried to completion. By manipulation of the guys and the anchor bolts through the spherical bearings, it was a simple matter to bring the trusses to closure at the center. Riveting was begun as soon as it was assured that the next adjacent frame was closing properly.

Having completed the two interior frames, the south intermediate and south exterior frames were completed in order, after which the traveler was taken through the tower to the north intermediate and north exterior frames. The filling in of the small steel, including the rods, was carried on by two other gangs simultaneously with the main erection.

Above the arches the stiffleg was discarded for a guyed derrick, which was raised to the successive decks, by the simple shifting of the guys. No particular difficulty was experienced until the section above El. 300 was reached, where outriggers were required for the guys. Above El. 364, a single gin-pole was left for the setting work, all pieces having been sent up to El. 400 before the removal of the guy derrick.

The total steel contract amounted to 1460 tons, and was detailed, fabricated and erected in 220 days. But little of the steel was to be had on the Pacific Coast, requiring the ordering and delivery of the major portion of the large sections from the East. The upset rods, clevises and turnbuckles were a further source of delay.

TIMBER ERECTION

By reason of the fact that the completion of the steel contract had preference over the timber construction, all of the general contract work, carried on simultaneously with the steel erection, had to be so performed as not to interfere with the steel contract. This resulted in the flat framing of a large percentage of the lower columns and trusses, far in advance of the time for the erection of the members in place. The tower site was at times severely congested, the framed material being piled high, in the inverse order to its erection.

When the steel erection had progressed sufficiently so as not to be interfered with by the timber, this framed material was rapidly swung up into place, either by raising shears or by tackle hooked directly into the steel frame. The heaviest single hoist in timber was the large four-post trussed column at the corners of the main-tower frame. These columns were 5 ft. by 7 ft. in plan, 150 ft. in length, and contained 16,800 ft. b.m., which, with the iron fittings, weighed approximately 35 tons. These columns, as all other large parts, were framed with their feet in approximate position, so that in raising only holdback lines were necessary to guide them into position.

Above El. 152 a 70-ft. Chicago boom was used, its foot being set at El. 300 and its topping lift at El. 364. This was very efficient in setting the major portion of the upper construction. Of the 1,500,000 ft. b.m. in the construction of the tower, two-thirds were below El. 152. Of the third above, the greater portion was in the form of floor material and wall studding, requiring but little framing out of place. There were few heavy pieces, therefore, to be moved with this rig.

As a result of the procedure of the steel contractor, it was possible to practically complete the timber framing of the east colonnades and the basins of the east court while the corresponding area to the west of the tower was being utilized as a storage yard for the steel. When finally this was available, the general contractor carried on the framing of that portion simultaneously with the construction of the upper portions of the tower proper. Three parts of the work, entirely independent of each other, were therefore worked upon and brought to completion together.

The contract time on the general contract, which included the foundations, the timber construction, the ornamental staff and plaster and the high-pressure and electrical installations, was originally 200 days, 120 of which were guaranteed to be after the completion of the steel contract. The overlapping of the two contracts, however,

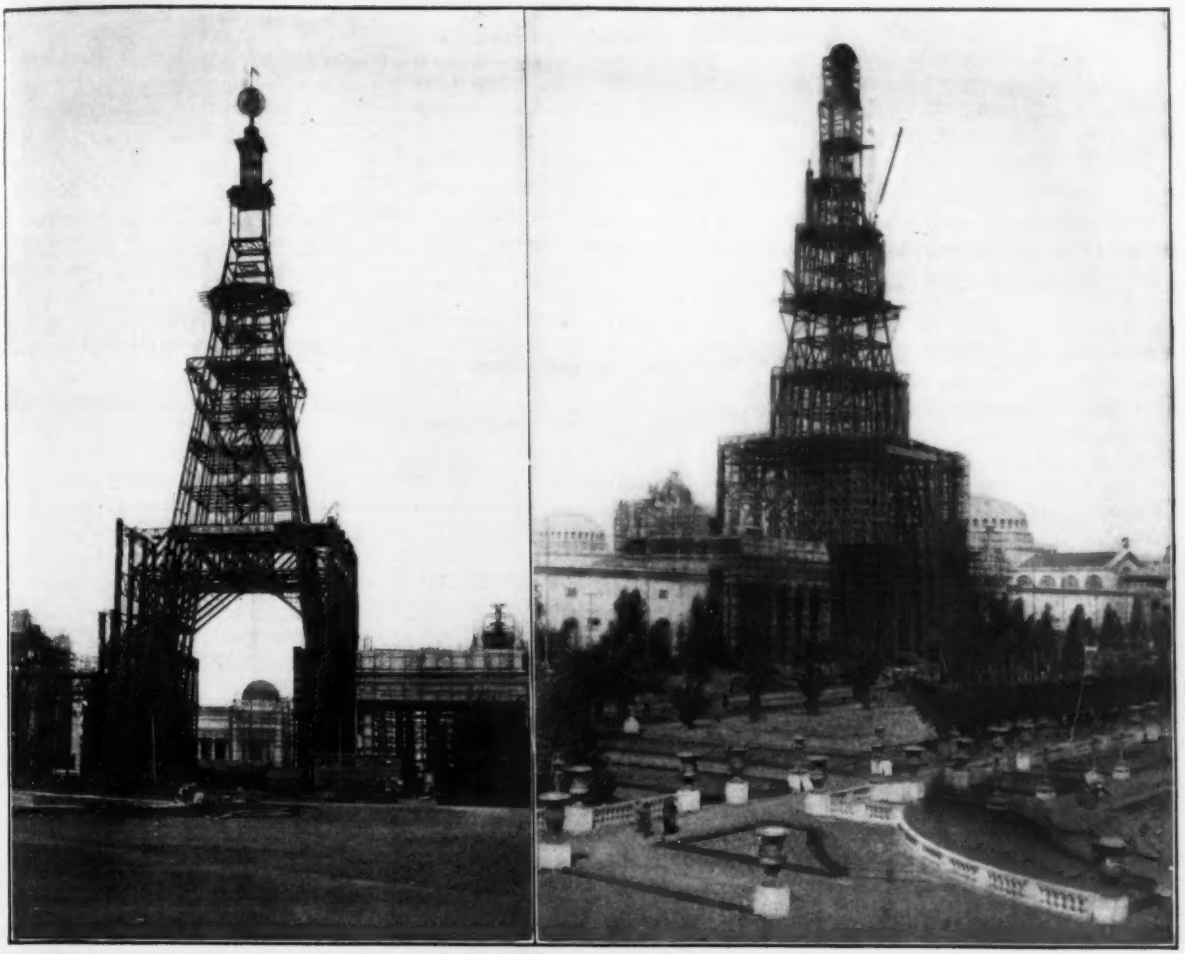


FIG. 4. STEEL FRAME ALONE AND WITH COMPLETE TIMBER FRAMING

operated to extend the total time on the general contract to approximately 270 days.

FIRE-PROTECTION

In view of the imminent risk from fire in a structure of such inflammable construction, it was early decided that spectators should be excluded from the building. For this reason no attempt was made toward fireproofing either the elevator shaft or the stairways.

Other fire-protection consists of high-pressure monitor standpipes and cornice sprinkler systems, under a maximum static head of 320 lb. per sq.in. The monitors are required to deliver smooth solid streams of 1500 gal. per min. through a 2-in. tip. A test pressure of 400 lb. per sq.in. is to be maintained for a period of 15 minutes without excessive leakage. Sprinkler nozzles are installed at all cornices and so set as to throw either fan-shaped or continuous arc jets. These result in a possible complete water curtain over the entire outer surface of the building. The outstanding portions of the tower are isolated from the adjoining buildings to either side by continuous fire-walls of reinforced concrete, 60 ft. high and forming a parapet wall above the level of the adjoining roofs.

The timberwork in general may be characterized as slow-burning mill construction, as few timbers of less than 8 in. exposed dimensions were permitted. Six-inch faces were used only where absolutely necessary. These remarks do not apply to small furring lumber entirely closed in plaster and staff, as the fire-resisting properties of the latter are ample.

TOTAL COST AND UNIT COST

The total cost of the structure and its attached colonnades, taken from the latest recapitulation, is $420,000, as tabulated herewith:

Piledriving	$6,600
Furnishing and erecting steel	137,400
General construction	216,000
Models	18,200
Lumber	22,700
Decorative painting	4,000
Further electric wiring not in general contract	2,000
Further plumbing and piping not in general contract	1,500
Paving	500
Incidentals	11,100
Total	$420,000

In giving the unit cost of the steel frames, the volume of the structure is taken as the volume of the tower, that is, excluding the two side colonnades. On this basis the tower contains 3,440,300 cu.ft., and the 1403 tons of steel amount to 0.816 lb. per cu.ft., which cost in place $0.04 per cu.ft.

Including the colonnades, the tower contains 3,907,500 cu.ft. and its total cost was $0.1067 per cu.ft. of inclosed content.

PERSONNEL

The plans were prepared by the Division of Works of the Exposition company; H. D. H. Connick, Director of Works, and A. H. Markwart, Assistant Director of Works.

The structural work and substructure were designed by the writer and others under the immediate supervision of H. D. Dewell, Chief Structural Engineer. The architectural design was by Thomas Hastings, of New York.

The first design of the structural part of the tower was made by J. D. Galloway, Consulting Engineer, of San Francisco, the writer acting as his principal assistant. At that time the program of conditions was more ambitious. It provided for a continuous fireproof wall between the main tower and adjacent parts, four elevators in the lower portion, and three fire-resisting ceilings at El. 154, 255 and 336, as a means of temporarily localizing a conflagration. Furthermore, the several decks were completely floored and designed for a total load of 150 lb. per sq.ft., the intention being at that time to use the tower for observation purposes. Steel framing was used in other parts, such as the four corner pylons of the first main deck.

On taking preliminary prices and making estimates for the completed plans, it was found that the cost of this design exceeded the appropriations and that the program of requirements must be revised—structural as well as the architectural. The changes were so numerous and the necessity of reducing the cost so apparent that a new design was made, as noted above.

The steel contractors were Dyer Bros., of San Francisco, while the general contract, including all except the steel frame, was executed by Commary-Peterson Co., Inc., also of San Francisco.

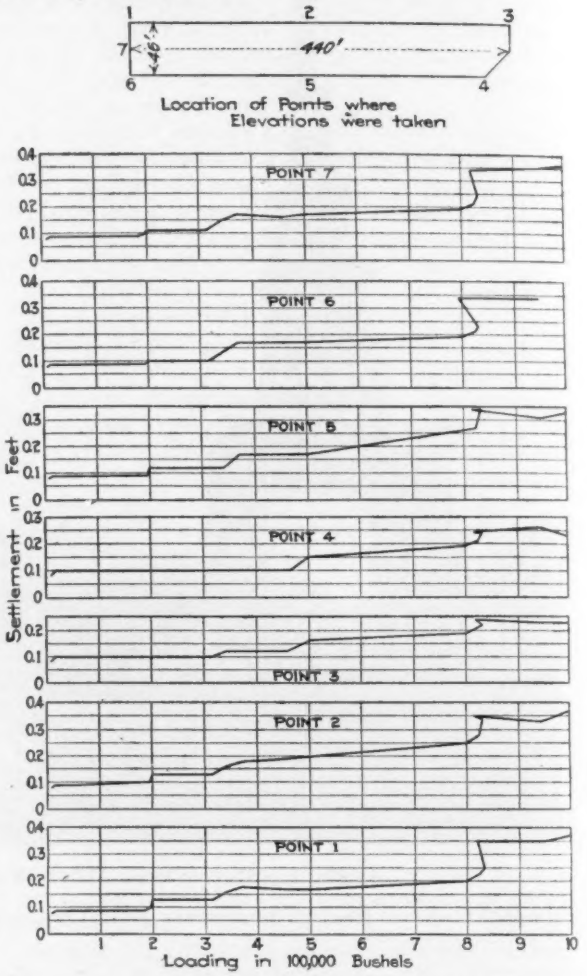
Settlement of Two Grain Elevators

In two reinforced-concrete grain elevators of recent construction, built on clay soil, settlement has been observed. The first case was not serious, but is of interest as we are able to give diagrams of the recorded movements. In the second it was necessary to restrict the loading of the bins.

The first elevator referred to is that of the Kansas-Missouri Elevator Co., in the east bottom, at Kansas City, Mo. This was designed and built by the Stephens Engineering Co., of Chicago, which also installed the machinery equipment. It was commenced Apr. 29, 1914, and completed Aug. 11. It is 440x46 ft., with a height of 111 ft. over all. Its capacity is 1,139,000 bu. and it is adjacent to an older elevator of 1,000,000-bu. capacity.

The excavation of the site was done by steam shovel, and amounted to 6900 cu.yd. From test holes sunk in the vicinity it was not considered necessary by the engineers to use piling with the slab type of foundation and unit construction of bins. A 22-in. slab of reinforced concrete was first placed in the excavation. Upon this were built the foundation walls, 15 in. thick and approximately 8½ ft. high, and on top of these is a 12-in. slab upon which the bins rest. The footing slab was reinforced as a beam under the foundation walls in order to take up the concentrated load at those points.

There are 40 bins, 37 of which are 22 ft. inside diameter; the others are slightly smaller to permit of the necessary track arrangement. These bins are 84 ft. high and have over the entire top a 7½-in. slab of reinforced concrete, which constitutes the distributing floor and roof. The walls are 7 in. thick, reinforced in both directions with ½-in. rods. The bins are so constructed as to have one wall in common at the points of contact. The reinforcement at this point is lapped, and extra bars are placed to tie the bins together. The structure comprises 9823 cu.yd. of concrete.



DIAGRAMS OF SETTLEMENT OF THE KANSAS-MISSOURI GRAIN ELEVATOR AT KANSAS CITY, MO., DURING THE FILLING OF THE BINS

The levels to determine the amount of settlement were taken on the slab that rested on the foundation walls and upon which the bins were placed. They were taken every week, or oftener if it was thought that there had been any unusual settlements. In this way it was possible to ob-

TABLE OF ELEVATIONS AT OBSERVATION POINTS SHOWING SETTLEMENT AND FLUCTUATIONS OF THE KANSAS-MISSOURI GRAIN ELEVATOR

1914 Date	Point 1	Point 2	Point 3	Point 4	Point 5	Point 6	Point 7
June 8.....	100.96	100.96	100.96	100.96	100.96	100.96	100.96
Aug. 15.....	100.88	100.88	100.88	100.88	100.88	100.88	100.88
Aug. 22.....	100.87	100.87	100.86	100.86	100.87	100.87	100.87
Aug. 29.....	100.87	100.86	100.86	100.86	100.87	100.87	100.87
Sept. 5.....	100.86	100.86	100.86	100.86	100.87	100.87	100.86
Sept. 10.....	100.84	100.84	100.86	100.86	100.84	100.86	100.85
Sept. 14.....	100.83	100.83	100.86	100.86	100.84	100.86	100.85
Sept. 26.....	100.83	100.83	100.86	100.86	100.84	100.86	100.85
Oct. 3.....	100.80	100.80	100.84	100.86	100.84	100.82	100.81
Oct. 10.....	100.78	100.78	100.84	100.86	100.79	100.79	100.79
Oct. 17.....	100.79	100.77	100.84	100.86	100.79	100.79	100.80
Oct. 24.....	100.79	100.76	100.80	100.81	100.79	100.79	100.79
Oct. 31.....	100.78	100.75	100.81	100.80	100.75	100.78	100.80
Nov. 7.....	100.76	100.71	100.77	100.77	100.70	100.77	100.77
Nov. 14.....	100.73	100.68	100.75	100.76	100.69	100.75	100.75
Nov. 21.....	100.71	100.62	100.74	100.72	100.62	100.73	100.72
Nov. 28.....	100.61	100.61	100.72	100.72	100.62	100.62	100.62
Dec. 5.....	100.61	100.63	100.73	100.70	100.65	100.62	100.61
Dec. 12.....	100.59	100.59	100.73	100.73	100.63	100.60	100.60
Dec. 19.....	100.58	100.59	100.71	100.73	100.59	100.60	100.59

tain a record of the settlement at the different points as the loading increased or decreased, and these results have been plotted to show the fluctuations of the structure. The bins were loaded as uniformly as possible.

It will be seen by the accompanying diagrams and table that settlement did not occur equally at the different points, this being probably due to the way the bins were loaded. An interesting feature is that the structure, after settling a certain amount, seemed to spring back, showing a tendency to return to its normal elevation. This is held to show that the concrete and steel had not been stressed to a dangerous limit. Computations made as to the largest amount of allowable deflection showed that a deflection of approximately 6 in. in the middle of the slab could occur without failure taking place. The levels showing the amount of settlement were taken about every week, while the loading was carried on during the week and not necessarily at the time the levels were taken. For that reason, it does not seem possible to obtain any relation between the time that the levels were taken and the loading, except with reference to the loading, which at that particular time was already in the bins.

Another interesting feature is that at one time the elevator was lightened by 13,677 bu. and the deflection increased at the different points to various amounts, though in each case showed an increase. It is assumed that all further settlement has ceased and that what occurred was due probably to the condition of the subsoil on which the structure was built. It is thought that if piling had been used and driven down to rock, none of this settlement would have occurred. It has not in any way damaged the structure nor has any effect been noticed in the machinery.

In the second elevator the foundation consisted of a reinforced-concrete slab supported on 30-ft. timber piles, which were not driven to rock. Before the bins were fully loaded, settlement commenced, mainly near one corner, and here the filling of the bins was reduced to about 20% of the full capacity. Investigation showed that while the piles were in a body of stiff clay, there was a deep bed of quicksand between this and the rock, the surface of the latter being approximately 50 ft. below the ground. The compression and movement of this quicksand under the heavy load had caused the settlement. This amounted to a little over a foot at the southeast corner. The top of the new structure pulled away about a foot from the older portion at the north, and it was necessary to cut the connection between the two in order to prevent damage. There are cracks also at the south end of the new structure.

Various methods were considered for securing stability for the foundation, and the first tried was the injection of cement grout into the quicksand. The plan was to drive a 2-in. pipe to the required depth and then to raise it gradually while grout was forced through it by a compressed-air apparatus. The end was fitted with a conical point for driving, held in place by a stem projecting within the pipe. As the pipe was raised this point was left in place. About ten pipes were driven to rock, at 50 to 53 ft., and an average of five barrels of cement was pumped into these at depths of 38 to 53 ft.

The plan proposed was to carry on this process around the sides of the concrete-slab foundation, so as to form a species of concrete retaining wall to inclose the mass of quicksand beneath the structure. Difficulty was experienced in driving the small pipes to the required depth and in pulling them gradually as the grout was forced in. Borings were made later to ascertain to what extent the cement had affected the unstable material. The results were not considered satisfactory, and this experiment was abandoned. A later proposition is to drive steel sheetpiling around the foundation, to inclose and confine the unstable material.

Some Unsatisfactory Experience with Sand-Clay Roads

By Samuel H. Lea*

Within the last few years about 15 miles of sand-clay or top-soil roads have been built in Mecklenburg County, N. C., in compliance with a popular demand for this type of road. These roads have not come up to the expectations of the public, and many taxpayers are bemoaning the loss of the money that went into their construction.

During the past winter there was a heavy precipitation in this part of the state. The weather was particularly hard on sand-clay and top-soil roads, especially those that were recently built and have not had time to come to proper form. The long-continued rainy season caused them to break up under traffic in many places and to remain unfit for use until the roads dried out in the spring.

An investigation into local conditions discloses the following situation: The sand-clay roads about which complaint is made are comparatively new and have not had time to become thoroughly compacted and settled. A sand-clay road, after completion, requires considerable time to attain its best condition. The surface may be hard and smooth, but this may be only a crust with several inches of loose material underneath. A rainy season will cause the crust to soften and break through; then the road will become cut up and the work will be an apparent failure. This is especially true if the sand and clay were not thoroughly mixed, as was said to be the case in several of the roads in this county.

In some instances the roads were built upon a clay subsoil, with no provision for subdrainage. In places where soft spots developed on these roads, storm water collected and remained until evaporated, causing mud holes that increased in extent with each succeeding rain.

Perhaps the most important feature in the situation is the lack of maintenance since completion. It appears that there was no systematic plan of road dragging, and that long stretches of road were not dragged at all. It is in such places that the worst conditions obtain.

It is evident that too much was expected of this type of road. The macadam roads in some portions of the county have remained passable for years with very little maintenance work. With a large mileage of macadamized roads in the county that require only infrequent attention, the necessity of active maintenance work was not fully realized. It is quite probable that after the sand-clay roads in this county are worked over and dry out thoroughly they will show up all right.

Pavement Guarantees in Philadelphia—(a) Pavement paid for wholly by the city—granite block, one year; asphalt, three years; wood block and vitrified block, five years. (b) Pavements of any character paid for in whole or in part by assessment bills—five years.

*City Engineer, Charlotte, N. C.

River-Front Improvement at Cape Girardeau, Mo.

The St. Louis & San Francisco Ry. has reclaimed a stretch of land of about two acres along the Mississippi River at Cape Girardeau, Mo., in order to straighten its line, which originally followed the foot of the bluff around a deep indentation in the river bank. The improvement not only gives a more direct line and eliminates some sharp curvature, but also provides additional space for yard development. A new passenger station is to be built also, and part of the new land is deeded to the city for a public park. By agreement with the city the station is to cost not less than $35,000.

FIG. 1. BUILDING A CONCRETE AND RIPRAP WALL FOR RECLAIMING LAND ALONG THE MISSISSIPPI RIVER AT CAPE GIRARDEAU, MO., ST. LOUIS & SAN FRANCISCO RY.

The reclamation work consisted in building a river-front wall about 1600 ft. long to cut off the indentation of the shore line, and then filling the inclosed space to grade. The end portions of this wall are of concrete, founded on solid rock, but this would have been very expensive construction at the central and deepest portion, and for that portion (about 600 ft.) a riprap toe wall was used with a paved slope to the top of the fill.

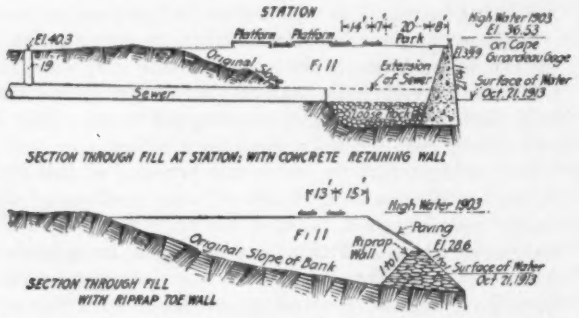


FIG. 2. CROSS-SECTIONS OF FILLED GROUND AT CAPE GIRARDEAU, MO.

The concrete wall has a maximum height of 36 ft.; its base width is equal to half the height, and the top width is 3 ft., while the river face has a batter of 1 : 12. The top of the wall is at elevation 39.92 or about 3½ ft. below the high-water level of 1903. For 600 ft. opposite the station the wall will have a parapet and sidewalk as shown. The riprap toe wall is 6 ft. wide on top, with side slopes of 1 : 1 and a maximum height of 20 ft., except that at the end it is sloped up to 36 ft. to meet the concrete wall. The same riprap stone is used for paving the slope of the fill above the toe wall, 1 to 1.5 slope.

The concreting was done from a plant on shore, the mixer discharging into the bucket of a 76-ft. elevator tower, and the concrete being distributed by steel chutes. The first section of the chutes was suspended from a boom on the tower, while the remaining sections were carried by falsework bents. For the bottom portion of the walls the concrete was set in place in 14 ft. of water by a drop-bottom bucket handled by a derrick scow. This prevented loss of cement which would occur by dumping concrete through the water. The concrete was spouted 275 ft. to the bucket. For the upper portion of the work the concrete was spouted directly into the forms. The concreting tower is shown in the background in Fig. 1.

For the footing course, the coffer-dams were 16x16 ft., in sections 56 ft. long; these were floated into place on barges, and then picked up and set by a derrick scow. The section was then sheeted and the mud pumped out with a centrifugal pump so as to give a clean rock surface for the concrete. The end of the adjacent form was torn out, the ends of the concrete sections being united by keys. The lower part of the wall was built in this way up to the water level, and upon this were erected the forms for the neat work, which was built in 48-ft. sections. These forms were of 2-in. tongue-and-groove dressed lumber, with posts 4x6-in. spaced 4 ft. c. to c.

About 300 ft. of this concrete wall runs through the toe of the original bank of the river, with a maximum cut of 25 ft. deep. This is in clay and old riprap, and the excavation was done with a clamshell bucket handled by a derrick scow. The rock taken from this excavation was used in the riprap toe wall.

The stone for the riprap wall was obtained from a quarry ½ mile distant and was hauled in end-dump wagons. These were driven onto barges which floated them

to the site so that they could be dumped directly in place, the stone being then dressed to the proper slope by hand. The fill was made by dumping from cars on a temporary track that was moved outward as the filling progressed.

The work is under the direction of F. G. Jonah, Chief Engineer of the railway, and H. B. Barry, District Engineer; L. L. Kerns is Engineer in Charge. The work is being done by the J. W. McMurry Contracting Co., of Kansas City, Mo. The estimated cost is $240,000, including the track changes and the new station.

A Graphic Recorder for the Cole Pitometer

The automatic water-flow recorder shown in the accompanying illustrations has been developed as part of the well-known Cole "Pitometer" system,* so that this can be used with accuracy and reliability by any waterworks superintendent or engineer.

This new recorder is a circular-chart curve-drawing differential pressure gage of unusual sensitiveness and

FIG. 1. INTERIOR (CLOCK COMPARTMENT) OF GRAPHIC PITOMETER RECORDER

accuracy at high and low velocities. Such primary requisites before were secured only with the photographic recorder.

The upstream pressure tube is connected to a multiple-diaphragm chamber and the downstream tube to the sealed case surrounding the diaphragms. Any movement of the latter therefore is proportional to the pressure difference at the Pitot tube, therefore to the velocity of flow and to the quantity passing. This motion is transmitted by driving links connected between the end diaphragm and a rotating magnet. Outside the pressure

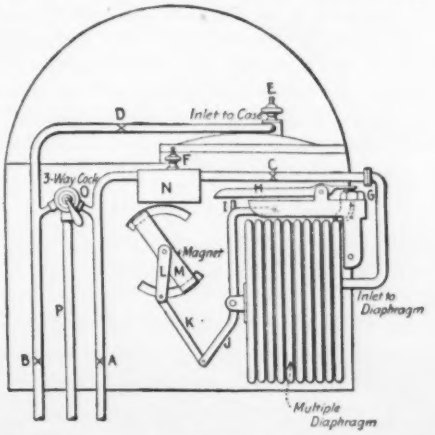


FIG. 2. ELEVATION OF DIAPHRAGM COMPARTMENT, PITOMETER RECORDER

case (inside the clock compartment) is a follower magnet which drives, through a multiplying gear, a spiral disk. A stop on the recording-pen arm is brought against the edge of this spiral once each 3 min. and then is lifted away by the clockwork. This is accomplished through a pawl engaging a long ratchet on the pen arm. The edge of the spiral which engages with the pen-arm

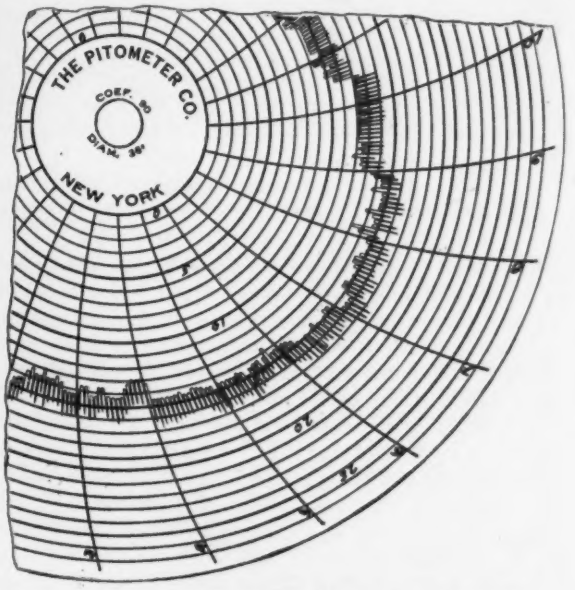


FIG. 3. SECTION OF THE CHART OF A GRAPHIC PITOMETER RECORDER

stop is finely toothed near the zero end to insure definite points of contact.

The chart produced by this recorder is peculiar since the pen records the entire oscillatory motion of the arm. Only the outer points of the curve traced are read. The instrument is best adapted to station use, but can be set up for outside measurements.

The "Pitometer" system has been developed by the Pitometer Co., 220 Broadway, New York City.

*The "Pitometer" system comprises a Pitot-tube current meter for accurately measuring flow of water in pipes, an automatic photographic recorder (or a substitute portable manual recorder made up of a U-tube, chart drum and punch marker), and various accessories for the convenient study of flow in the various mains of a water-works. The skill required to use the photographic recorder has heretofore somewhat limited the application of the system.

An Explanation of Some Common Painting Troubles

An accurate description of some common defects of painted surfaces, with suggestions as to the causes and means of avoiding these defects, was given by Gustave W. Thompson, Chief Chemist of the National Lead Co., New York City, before the last annual convention of the Maintenance of Way Master Painters' Association. The following brief notes are extracted from this description:

Checking and Alligatoring

Checking and alligatoring (see Figs. 1 and 2) consist in the development of fine interlacing lines on the surface of a paint; if the meshes are small it is called checking; if they are large the action is called alligatoring.

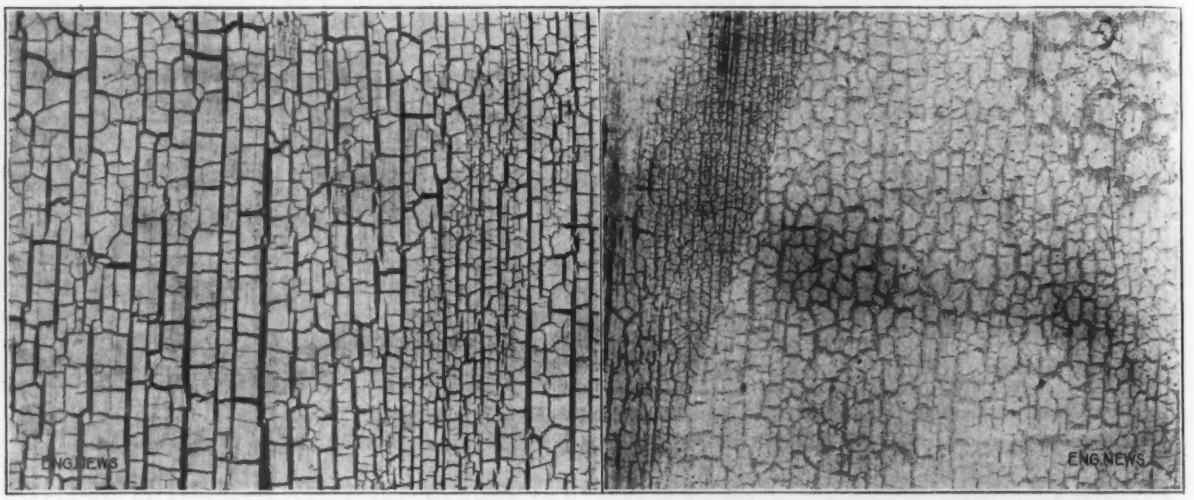
If varnish is applied over a grained surface the ground of which is a yellow ocher paint which will not harden properly, the surface is quite sure to alligator. In such a case, while the first coat was sufficiently dry at the time the second coat was applied to permit the application of the second coat, it had not reached what might be considered a fit condition of dryness; that is, if the second coat had not been applied and the first coat had been exposed to the air, the hardening which is due to oxidation would have continued. The second coat, on the other hand, can keep on oxidizing and hardening indefinitely. The shrinking in volume of the second coat during this operation of oxidation and hardening demonstrated by the investigations of Prof. A. H. Sabin produces one of two effects—rupturing of the outer coat with consequent alligatoring or checking, or the outer coat becoming thinner without rupturing. Which of these effects occurs depends upon the under coat.

This action occurs whenever a paint is applied over another that inherently will not dry hard—as in the case of a harder paint applied over an asphaltum paint; very extreme examples occur where a nonvolatile mineral oil has been used in an under coat. Mr. Thompson has experimentally produced a series of paint defects beginning with very fine checks on one side and ending up with very large alligatoring on the other side, which shows that alligatoring is the same as checking except for greater difference in hardness between first and second coats (see Fig. 2).

In order to avoid checking and alligatoring it is best to have the under coat as hard as is practicable, much harder relatively than the outer coat. Probably the most important thing to do is to allow as much time as possible between coats.

Cracking and Scaling

Cracking is the formation of hairlike lines, which extend practically through the paint layer down to the foundation (checking or alligatoring is superficial). In case of paint on wood which contains moisture, if the paint (in drying) contracts across the grain less than the wood shrinks, there will be a bulging and consequent cracking of the paint with the grain. As wood does not shrink much endwise, contraction of the paint with the grain may cause cracks across the grain.



White Paint on Ochre Priming — Composite Paint on Oily White Lead Priming

FIGS. 1 AND 2. CHECKING AND ALLIGATORING OF PAINT ON WOODEN TEST PANELS

(Fig. 2 shows that the mesh size of the checks may range from very small to large due to small changes in the physical condition of the paint coats)

When cracking occurs the paint has very little elasticity, is very hard, and is high in tensile strength. The size of the spaces inclosed between the cracks is largely dependent upon the tensile strength of the paint.

As soon as cracks occur there is an opportunity for moisture to enter the wood. This moisture travels under the paint film, exerts a pressure against the paint film, the bond beween the film and the wood is weakened, and scaling results.

It is noticed that cracking and scaling take place most often when the paint film is thick. One method of prevention, therefore, is to avoid the accumulation of paint coats upon the surface.

It is obvious, of course, that those pigments which give hard, strong and contracting coats should be used only in moderation. There are some cases, however, where the painter is in a dilemma. If he wants to avoid cracking and scaling he can do this only by allowing a certain amount of checking. Hard undercoats tend to prevent checking, but too hard coats generally favor cracking and scaling. Soft undercoats favor checking, but tend to prevent cracking and scaling.

Scaling of Paint on Galvanized Iron—Galvanized iron has been washed with acids, with alkalies, and with special galvanic primers, such as sulphate of copper, etc., without preventing scaling. The scaling on galvanized iron is evidently due to the impossibility of securing a proper bond. It is more apt to occur with hard or thick coatings than it is with more elastic and thin coatings.

Blistering and Peeling

Blistering and peeling are due to the water in the wood forcing its way through the wood to the surface, breaking the bond of the paint and forcing it away from the surface. Blistering is very apt to occur where moist painted wood is subjected to heat, such as the heat of a radiator. Sometimes the heat of the sun is sufficient to increase the vapor pressure of the water and thus produce blistering. Painters sometimes complain that boiled linseed oil favors blistering. If this is so it can only be due to the fact that boiled linseed oil gives a more impermeable paint.

One safe method of preventing blistering is to remove the cause; that is, the moisture in or back of the wood. The practice of applying backing paint to window frames and in general on wood apt to become moistened on the inside has for one of its objects the prevention of blistering and peeling.

Blistering of iron and steel is due to a different cause. It generally results from the formation of gaseous products under the paint, due, perhaps, to the corrosion of the iron. Sometimes peeling will occur on enameled surfaces which have been painted without proper care being used to roughen the surface so as to produce a proper tooth.

Water Blisters—Sometimes it has been found in the case of a recently painted house that after a heavy rainstorm large blisters (Fig. 5) make their appearance. These blisters are filled with water, the water being, as a rule, between the coats of paint. The undercoats of paint are in sound condition, and it is impossible to conceive of the water having come from the back of the paint. The probabilities are that the water passed through the outer coating of paint by some system of diffusion and collected under it.

Loss of Gloss and Chalking

Gloss is due to an excess of oil or binding material, which, lying over the particles of pigment, gives a smooth, even, glass-like surface. Such organic substances as are used for the binding material in paint are more or less unstable. They are subject to fairly rapid destruction when exposed to the influence of air, moisture and sunlight. Loss of gloss is due to this destruction; chalking is but a step further, whereby the binding material is destroyed and the pigment becomes loosened on the surface.

Another type of chalking occurs in the case of pigments that are physically unstable. Basic sulphate of lead, a fire-process pigment, seems to be peculiarly susceptible to the influence of moisture, changing its structure so as to approach the crystalline and coarse structure of sulphate of lead prepared by the wet process.

Washing

Washing of paint is characterized by the surface giving up certain water-soluble substances. The surface when moistened rubs up like soap; there is a collection of washed-down pigment at the footings of columns and similar conspicuous places. There are also noted on the ceilings of porches brown spots consisting of substances which have been dissolved out of the paint by moisture. Usually a surface which shows washing is found to be in excellent condition underneath. Washing seldom takes place until loss of gloss occurs. It is less common in the country than it is in large cities, probably due to the greater quantity of sulphur gases present in urban atmosphere.

Cracking and scaling; the longitudinal cracks with bulging are the primary feature. | Extensive scaling; cross-grain cracks characteristic | Blistering of top coat only. The blisters are full of water

Figs. 3-5. Cracking, Scaling and Blistering

Spotting

In the spotting of paint the spots, which may be small or large, are much lighter in color than the remainder of the paint, and show greater chalking than the body of the paint. Moistened with linseed oil they return to practically the original color of the paint, indicating that there had been no destruction of the binding material and indicating also that at these spots there had been some excessive destruction or loss of oil. Spots are very apt to appear along the edges of cracks in wood. Sometimes they appear over and around putty, indicating the location of nail holes. Spotting occurs earlier in two-coat work than in three-coat work.

If portions of the surface painted are not fully satisfied by the priming coat, some of the oil in the subsequent coats may be absorbed by these spots; more rapid oxidation and destruction of the oil, due to the excessive air under and back of the spotted places, may be another factor. Spotting is to be avoided by the proper filling of the pores of the wood; by allowing sufficient time between coats for weaknesses to develop that can be corrected by subsequent coats; by the competent workman using a

stiff brush for the under coat, thereby forcing the paint into the wood.

There is another kind of spotting due to the spattering of lime on wood before it is painted, which lime spots work through the paint and by destroying the oil show on the outer surface.

Yellowing Out-of-Doors

The yellowing of paint outdoors applies mostly to white paint. There are two known causes: One is the presence of heavy mineral oil, and the other is the presence of alkalies or alkaline material. Mineral oil, remaining unchanged while the linseed oil oxidizes, exerts a solvent action upon the wood, bringing to the surface gums and other soluble organic material. Linseed-oil soap, which is formed by the action of alkaline materials (soda ash, borax, silicate of soda) upon linseed oil, tends to become a very dark orange on exposure to the air, and paint in which the pigment is surrounded by an oil which has undergone such a change will appear as yellow.

Yellowing Indoors

Yellowing indoors takes place in the dark, and if the object painted can be exposed to the sunlight the yellowing disappears. In the case of tints they are changed; blue becomes green, and red (if not too deep) an orange, etc. The explanation which seems most satisfactory is this: Ammonia is more or less present in the atmosphere. Ammonia turns white paint yellow similar to the action of the fixed alkalies referred to in the yellowing of paint outdoors. Ammonia may be used in manufacturing establishments, may be used for cleaning, and may be generated from the glue size applied to plaster walls.

Gloss paints are more apt to yellow than flat paints. Linseed oil yellows more than poppy oil. The pigment protects the oil more in some cases than in others. The more impermeable paint is less affected by ammonia. A thin coating of varnish over a paint or a little varnish in a final coat will often prevent serious yellowing.

Discoloration by Dirt

In New Orleans they are troubled a great deal with mildew, but in addition to this they are troubled a great deal with the dirtying of paint due to dust. New Orleans within the last ten years has introduced a system of drainage which has resulted in the earth becoming very much drier than it used to be. During the dry season the soil is dry to a greater depth than formerly.

Experiments conducted at New Orleans indicate that the quality of the linseed oil used there affects considerably the tendency of paint to become discolored both from mildew and from dirt. La Plata linseed oil when used in paint in New Orleans occasions the discoloration of the paint more than North American linseed oil. North American oil gives a harder coating than does La Plata oil and has a higher iodine absorption. Only the best quality of linseed oil should be used there and in similar places, and all linseed oil substitutes made from the semi-drying or nondrying oil should be avoided.

Mildew

Discoloration of paint on account of mildew is fairly common in the South. The ordinary pigments used do not seem materially to prevent the growth of mildew. Mildew grows better in elastic paints than it does in hard paints; moisture and shade are favorable to its growth. Experiments indicate that the mildew is killed by the presence of very small quantities of mercury compounds, and that a minute percentage of mercuric chloride added to paint will prevent mildew.

Sulphur

Discoloration due to the presence of sulphur compounds occurs with those paints containing lead pigment. It appears as a brownish or grayish metallic film over the surface of the paint. This film is a sulphide of lead and may be readily destroyed by peroxide of hydrogen. Sometimes this discoloration is due to the presence in paint of sulphides of the other metals, particularly sulphide of zinc (in lithopone).

The discoloration of lead compounds due to sulphur compounds in the air is not as common as one would be inclined to believe from what has been said upon the subject, and it is not usually permanent. Very frequently it will appear in a dry season and then will entirely disappear after a good rainstorm, due to the fact that sulphide of zinc is rather unstable and tends to oxidize to sulphate of lead.

There is only one method of preventing the discoloration of paint by the action of sulphur compounds in the air. That is to prevent the presence of such compounds, for the number of lead pigments which it is necessary for the painter to use is so great that it would seem impossible effectively to prevent discoloration in any other way.

New Heat-Insulating Material

A new fibrous form of silicon oxycarbide of value as a heat insulator has been developed by E. Weintraub of the General Electric Co., Lynn, Mass. It is a soft, resilient, fibrous material which after gentle compression returns to its original shape. It can be cut into plates, sheets and rings, which are self-supporting. It is an agglomeration of exceedingly fine fibers, the diameter of each being about 0.0006 mm., or 1/20 the diameter of a fine cotton fiber. The apparent density of the unworked material varies somewhat, being about 0.15 to 0.18 lb. per cu.ft. The actual density varies from 1.84 to 2.20, according to the variance of the composition from the theoretical $SiCO_2$.

The name "Fibrox" has been given to this product. It appears to be the best heat-insulating substance known. When compressed to 0.8 lb. per cu.ft. it is about twice as good as wool compressed to a density of 12 lb. per cu.ft., and equal to eiderdown at a density of 5 lb. per cu.ft. The heat-insulating value is approximately halved at a rise of from 100° to 500° C. At temperatures above 1000° C. in air it oxidizes slowly, being converted into silica. The heat capacity is very small on account of the very small mass of solid material (0.1 to 0.5 per cent. uncompressed). The material of "Fibrox" is a fair electrical conductor, but if sparking takes place at the contacts it is oxidized to silica, and this introduces an insulating layer. The specific resistance of the material itself is about one ohm per centimeter cube at ordinary temperatures, but that of the aggregation of fibers is about 360 ohms per centimeter cube.

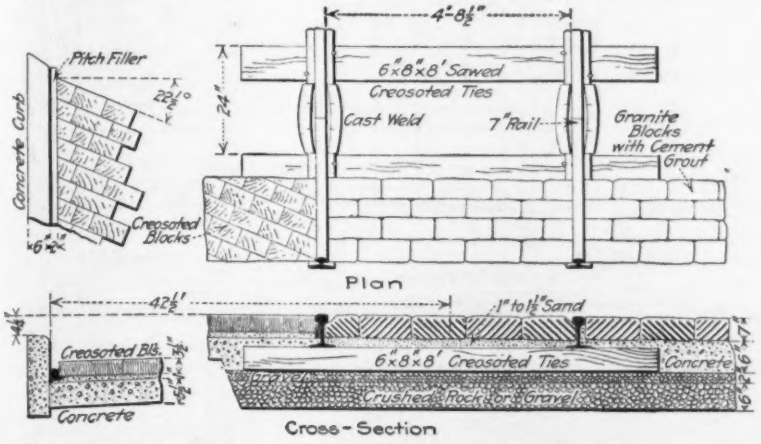
In the preparation of "Fibrox" a sealed muffle pot of clay-lined graphite has its inner bottom covered with pieces of silicon and a catalytic agent like calcium fluoride. After a few hours in a furnace at from 1400 to

1500 deg. C. the muffle is found filled with the material. It is explained that this product is a slow, quiet growth resulting from the contact of silicon vapor and carbonic-oxide gases, the one coming from the silicon in the muffle and the others from diffusion of furnace gases through the muffle walls.

Creosoted Wood-Block Paving Practice in St. Paul, Minn.

The twin cities of St. Paul and Minneapolis have had many years of successful experience with wood-block pavement, and at the present time this is the most popular type of pavement in those cities. They are laid by municipal day-labor, so that the cost has been reduced to compare favorably with sheet-asphalt pavements laid by contractors.

The accompanying drawings show the construction details of the creosoted wood-block pavement on University Ave., St. Paul—a 4-mi. stretch, 69 to 95 ft. wide,



CONSTRUCTION DETAILS OF UNIVERSITY AVE. CREOSOTED WOOD-BLOCK PAVEMENT, ST. PAUL, MINN., AS RECENTLY ADOPTED

containing 128,786 sq.yd. exclusive of the car tracks—and illustrate the most recent practice of that city. This is one of the largest single pieces of wood-block paving ever undertaken by the city, and the cost was $2.60 per sq.yd.

The pavement has an 8-in. crown on the 69-ft. width and a 10¼-in. crown on the 95-ft. width. On the wide section there are four car-tracks for a part of the way. The method of laying the blocks differs from standard practice in placing the courses at an angle of 22½° to the curb instead of at a right-angle. This necessitates specially cut half-blocks (in every course) along each edge of the pavement, but helps to take care of the breaking of the joints. This method of laying the blocks, presumably, is an additional safeguard against expansion troubles.

A 2-in. expansion joint filled with paving pitch is placed along each curb, but there is no expansion joint against the rail, which has a wood web-filler, as shown in the accompanying cross-section of the pavement. The pavement as laid last year has a double course of header blocks laid longitudinally next to the expansion joints at the curbs, but the design subsequently adopted, as shown in the sketch, omits these headers.

The blocks are 3½ in. deep and are laid on a 1-in. sand cushion, except in depressions or under viaducts, where sand and cement in the proportion 1:5, mixed and spread dry, is used as a cushion. The city authorities are also experimenting with a dry mortar cushion in the area 2 ft. each side of the street-car rails to give additional rigidity to this portion of the pavement. The foundation of the pavement laid last year is 5 in. of cement concrete, but the new design calls for 5½ in.

The paving work of St. Paul is done by the Department of Public Works, of which M. N. Goss is Commissioner and Oscar Claussen, Chief Engineer.

Economy in the Use of Creosote Oil

The economical use of creosote, in order to offset the effect of the war in Europe upon the creosoting industry in this country, was noted in a paper read recently before the Western Society of Engineers (Chicago) by H. F. Weiss, Director of the U. S. Forest Products Laboratory at Madison, Wis. He pointed out that of the 110,000,000 gal. of creosote oil used annually in this country, 70% comes from abroad.

Some of the creosoting companies applied to the laboratory for advice when the supply of oil failed. The advice given was to use less creosote (treating ties with about 2 lb. per cu.ft.) or to use zinc chloride, instead of shutting down the plant and giving no treatment. It was pointed out, however, that for satisfactory results a 2-lb. treatment must be handled in a different way from a 10-lb. treatment. We learn on inquiry, however, that it is not known that any companies have made a practice of treating ties with as low an absorption as 2½ lb. of creosote per cu.ft. except in the Card process, where the creosote is reinforced by ½ lb. of zinc chloride.

The authorities of the Forest Products Laboratory point out that one way to get good penetration with such a low absorption would be to dilute the creosote with some suitable oil, such as crude petroleum. With a mixture containing 3 parts of crude oil to 1 of creosote an injection of 10 lb. per cu.ft. would require but 2½ lb. of creosote. In this way it would be possible to get a good distribution of the oil and prevent rapid evaporation and loss of the preservative. The cost would be considerably lower than if 10 lb. of straight creosote were used, and if the practice became general the demand for creosote would be considerably lessened. While no service tests have been made as yet on timber treated with such a mixture, a consideration of the qualities necessary in a wood preservative makes it seem possible that satisfactory results might be obtained.

The Dredging Plant of the City of Philadelphia began work on Apr. 13 on the deepening of the channel in the Schuylkill River. Just previously the plant, which is operated by the Department of Wharves, Docks & Ferries, was engaged in deepening and widening the Frankford Creek. This was completed Apr. 1. On the new work the material taken from the river bottom will be pumped to the "Cannon Ball Farm," a property situated on the south side of the river. It is rumored that when this property is filled in, the department of Public Works will use it as a site for a city sewage-disposal plant. Dredging operations will continue until ice closes the river.

A Test of Filter-Rate Controllers

By Edward E. Wall* and Gurdon G. Black†

The St. Louis Water Department recently concluded a series of tests to determine the relative accuracy, reliability and merit of design of rate controllers for the Chain of Rocks filters. This is believed to be the first time that a careful and elaborate test of rate controllers has been made.

The specifications require that the controllers shall be capable of maintaining constantly any desired rate of flow through them between the limits of 1,500,000 and 5,000,000 gal. daily, although the difference in head of the water on the two sides of the controller may vary from 1 to 14 ft. Also, that between the limits of 1,500,000 and 2,500,000 gal. daily the rate of flow shall not vary from the figure at which it is set by more than 3%, and between the limits of 2,500,000 and 5,000,000 gal. daily the variation must not exceed 1½%.

The general arrangement for the test is shown by Fig. 1 and explained in some detail by the accompanying note.

Under the specifications each controller was required to have a registering device recording its output as measured by the controller itself. Two checks, one by disk meters and one by weir measurement, were provided against the controller record. The method of operation was as follows: Starting with the filter box full, giving a head of 12 ft. on the controller, the rate of pumping was regulated so that the water was gradually lowered in the filter box, thus producing in effect the loss of head that would actually occur in operating the filters, although the change in head was brought about much more rapidly than would usually occur in filtration. For example, the loss of head in a 50-min. run during the test might amount to as much as would occur in 24 hr. of actual filtration.

When the water in the filter box had been lowered until the head on the controller was at the minimum of 3.3 ft., the rate of pumping was increased so that the water began to rise and continued to do so until the maximum head of 12 ft. was reached, thus testing the controller on an increasing head.

Two controllers, called A and B, were tested, both of which utilized the principle of the Venturi tube for regulating the valve opening on the outlet. Controller A was connected to a Venturi tube 24 in. in diameter at the head and 10 in. at the throat, joined by a short 90° curve to a 24-in. valve, hydraulically operated, whose movement regulated the rate of flow. Controller A consisted of a cast-iron cylinder about 20 in. in diameter and 24 in. long, divided into four compartments, as shown by Fig. 2 and as explained in detail in the note below that illustration.

In Controller B the rate of flow was regulated by the movement of a butterfly valve consisting of two 12x12-in. vanes hinged in the middle and operated by spur gears engaging in a double ratchet connected direct to a hydraulic cylinder. Water from a pressure line is admitted into this cylinder through the movement of a pilot valve that is operated by a lever moved by floats in 6-in. tubes connected by 1¼-in. pipes to the head and throat of a Venturi tube, 24 in. in diameter at the head and 10 in. at the throat. These floats also operate the mechanism which records the rate of flow.

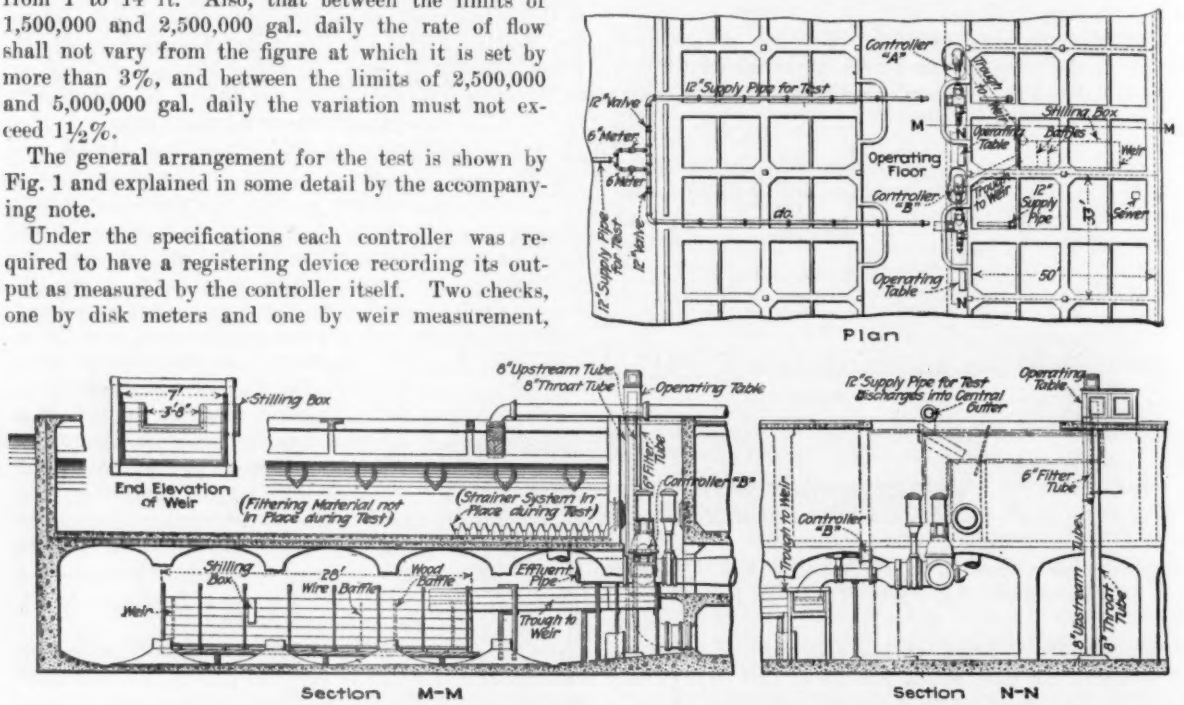


Fig. 1. Arrangement of Apparatus for Filter-Controller Test at St. Louis, Mo.

(A motor-driven centrifugal pump with a capacity of 3200 gal. per min. was installed on the edge of Basin 5, discharging through a 12-in. cast-iron pipe. Two 6-in. Worthington turbine meters were placed in parallel on this line, which was then branched into two 12-in. pipe lines leading to two filter boxes. The area of each filter box is about 1480 sq.ft., excluding the central gutter. No gravel or sand had been placed in either box, so that a variation of about eight feet in the elevation of water in the filter box could be had during each run. These elevations were read to the nearest 0.01 ft. The 24-in. effluent pipes of these filter boxes, on which the controllers to be tested were placed, emptied into flumes leading to a weir box 25 ft. long, 7 ft. wide and 2½ ft. deep, provided with two baffles. The first baffle was 5 ft. from the upper end of the box and was made of 2-in. lumber, perforated with 100 holes 2 in. in diameter. The second baffle, made of ordinary wire fly screening, was placed 3 ft. from the first. These baffles practically broke up all wave action due to the incoming water and gave a smooth flow to the 44-in. rectangular weir notch cut out on a ⅛-in. steel plate. Seven feet back of the weir, a stilling box 5 in. square was built, in which was placed a hook gage equipped with a vernier, reading to 0.001 ft., and a micrometer adjustment screw.)

*Water Commissioner and †Engineer-in-Charge, Supply and Purifying Division, Water Department, St. Louis, Mo.

The accompanying table gives the maximum, minimum and average rates of flow, with the percentage variation from the average, for the various rates of flow

TESTS OF FILTER-RATE CONTROLLERS, ST. LOUIS, MO.

No.	Date, 1914	Rate of Flow, Million Gallons per Day, Max.	Min.	Av.	Average Percentage Variation, Max.-Min.	Max.-Av.	Av.-Min.
		Results with Controller A					
2	Aug. 7	1.640	1.395	1.559	16.67	4.92	10.74
4	Aug. 7	1.690	1.483	1.610	12.86	4.97	7.88
3	Aug. 7	2.140	1.987	2.063	7.31	2.24	5.06
6	Aug. 8	3.070	2.990	3.035	2.63	1.15	1.48
5	Aug. 8	3.775	3.292	3.487	13.85	8.26	5.59
1	Aug. 7	3.965	3.865	3.916	2.55	1.25	1.30
7	Aug. 8	5.020	4.935	4.978	1.30	0.84	0.46
		Results with Controller B					
13	Aug. 1	1.575	1.470	1.520	6.91	3.62	3.29
12	July 31	2.087	1.987	2.040	4.90	2.30	2.60
9	July 27	2.975	2.907	2.942	2.31	1.12	1.19
7	July 28	2.975	2.915	2.937	2.04	1.29	0.75
8	July 28	4.020	3.957	3.987	1.58	0.83	0.75
10	July 29	4.987	4.959	4.959	1.65	0.56	1.09
11	July 30	6.155	5.890	5.999	4.42	2.60	1.82

for both controllers. The average was obtained by multiplying each observed rate by one-half of the time interval between the preceding and following observed rates, and dividing the sum of these products by the total time.

Preliminary tests on controller A began on July 20, but only the results of the final seven runs are tabulated. All of the tests run on controller B are given.

The rate of flow, as shown by the turbine meters, was found to be about 2% greater than that shown by weir measurement. The rate of flow over the weir was calculated by the Fteley and Stearns formula for all heads under 6 in., and by Francis' formula for all above this amount.

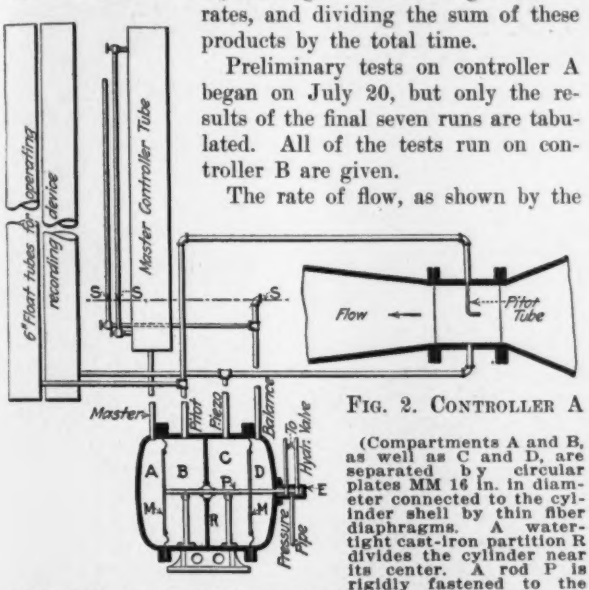


FIG. 2. CONTROLLER A

(Compartments A and B, as well as C and D, are separated by circular plates MM 16 in. in diameter connected to the cylinder shell by thin fiber diaphragms. A watertight cast-iron partition R divides the cylinder near its center. A rod P is rigidly fastened to the plates MM, but in passing through the partition E is connected to it by a small flexible diaphragm to eliminate the friction which would result if a stuffing-box were used. This rod P is connected to a pilot valve E which, with a small movement, admits water from a pressure line for operating the 24-in. valve which regulates the rate of flow.

Compartment B is connected by a ¾-in. pipe (Fig. 2) to a small pitot tube, located in the center of the throat of the Venturi tube. Compartment C is connected to the throat of the Venturi tube by a ¾-in. pipe, marked piezo in Fig. 2. Compartments A and D each have a vertical tube as shown in Fig. 2, called respectively "master" and "balance" tubes.

The operation of the controller is as follows: Compartments A and D are filled until the water rises to an arbitrary level SS in the balance and master tubes. Water is then allowed to enter from the Venturi tube, filling compartments B and C and rising in the pitot and piezo tubes to the height of the water in the filter box. The water in the master tube is then raised to a point above S equal to the calculated height the water in the pitot tube should exceed that in the piezo tube for the desired rate of flow. This excess head in the master tube moves the plates MM and also the pilot valve E admitting water from the pressure line to open the 24-in. outlet valve until the excess head of water in the pitot tube over that in the piezo tube moves the plates MM and the pilot valve back to a neutral position. Branch lines from the pitot and piezo tubes are connected to two 6-in. pipes, in which floats operate the mechanism recording the rate of flow.)

The tables show controller B superior in controlling the rate of flow, except at the nominal 5,000,000-gal. rate, when the variation is only slightly above that of controller A. Controller B has a uniformly decreasing variation in rate from the 1,500,000 to the 4,000,000-gal. rate, and an increasing variation for the succeeding rates above 4,000,000, while Controller A shows a decreasing variation up to 5,000,000 gal., with an erratic action on test No. 5, which characterized the machine throughout the entire time of preliminary tests for two weeks preceding the final tests.

Controller B was selected for the Chain of Rocks Plant. The tests were made under the supervision of the writers, with C. M. Daily, assistant engineer, water department, in direct charge of the work.

The Maintenance of Drainage Ditches

The maintenance of drainage canals and ditches in proper condition and their periodical cleaning to secure a free flow of water are permanent and troublesome problems to most engineers and superintendents of large drainage districts, owing to the difficulty of getting the authorities to appropriate funds for such work.

The Sny Island Levee Drainage District, in Illinois, affords an example of this problem. This district, which has existed for over 40 years, has an area of 110,000 acres, with 53 miles of levees along the Mississippi River. The maintenance of the internal canals or ditches has not been a serious matter until recent years, as the natural drainage facilities were so efficient. The tributary watershed lying outside of the district is about three times the area of the latter and comes to the main drainage canal with a fall of 10 to 50 ft. per mile, while this canal has a fall of only 0.4 ft. per mile.

For the past 10 or 12 years there have been small assessments grudgingly appropriated for straightening and cleaning out the main drainage artery, called the Sny, and there have been six independent subdistricts formed to give protection from local creeks only. About a year ago the district finished the expenditure of $160,000 in straightening and enlarging the Sny, and it is now expending $190,000 on the same work. This will probably be all the straightening to be done in the near future. For this work dipper dredges with 2½-yd. and 4½-yd. buckets have been used. In many places the full capacity of the dredges was required, so that other means must be used for future work.

The matter has been studied by C. G. Elliott, consulting drainage engineer, of Washington, D. C., and a comprehensive plan of drainage was recommended. Under this plan, the enlargement of the Sny channel would be continued and subdistricts formed to provide for local conditions, while the creek waters would be carried through settling basins to remove much of the silt before reaching the Sny.

It is considered by E. J. Chamberlain (Pittsfield, Ill.), Chief Engineer of the Sny District, that the annual removal of silt from the large channels and its occasional removal from the smaller drainage canals form an important problem to be faced by drainage districts that must provide for silt-carrying streams, and so far there has been no satisfactory solution as to economical and efficient work. For the larger canals, hydraulic dredges work well only with some classes of the materials found, but they may deposit the spoil over or around large spoil-

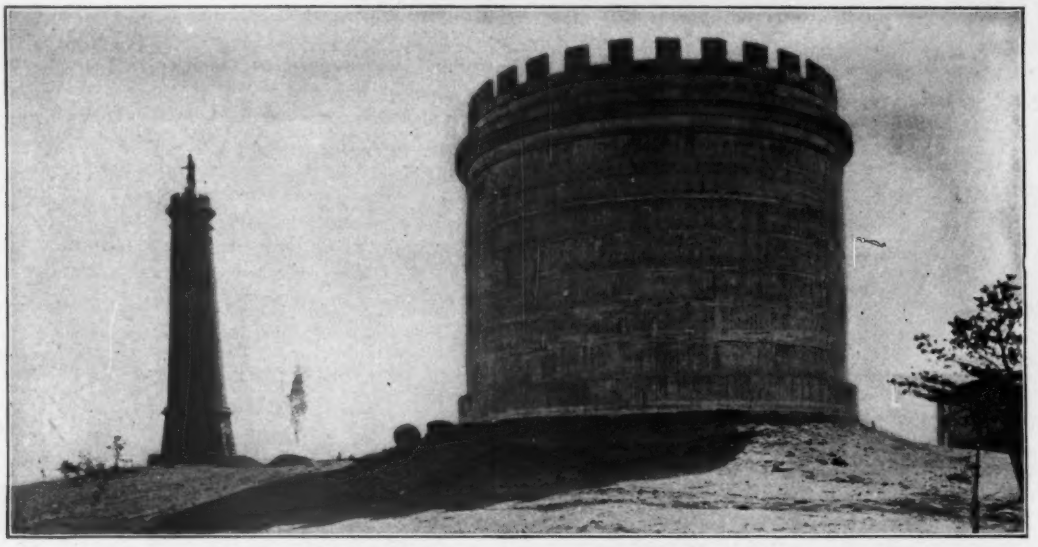


FIG. 1. DUXBURY, MASS., REINFORCED-CONCRETE TANK AND THE MILES STANDISH MONUMENT

banks left by dipper dredges. The experience with a small centrifugal-pump dredge has not been satisfactory.

It is believed that two types of machines need to be developed for this class of work—a floating machine that may be easily removed from one silt-bar to another and that will at a low cost per cubic yard remove the silt from canals with a bottom width of 40 to 150 ft., and a machine (preferably movable overland) to remove silt from ditches of 6 to 20 ft. bottom widths.

The Duxbury Reinforced-Concrete Water-Works Tank

One of the strongest arguments for adopting concrete for water-works tanks and standpipes is that a structure of that material is capable of the artistic treatment necessary to make it an adornment rather than a blot to the landscape. Where such a structure is to be the most prominent object on a hill already occupied by a monument distinguished for both æsthetic and sentimental reasons, the selecting of concrete is easily explained. Such is the case of the 330,000-gal., Duxbury, Mass., water-works tank illustrated herewith. Fig. 1 shows its prominence in the landscape and its proximity to the famous Miles Standish Monument.

STRUCTURAL DETAILS—Fig. 2 shows the main details of the tank, which is 40 ft. inside diameter and 35 ft. high. The bottom is a reinforced-concrete slab built in two 12-in. layers—the bottom one of 1:2:4 concrete and the upper one of 1:1½:3 mixture with the addition of 5% of hydrated lime. The walls are of 1:1:2 concrete with 5% of hydrated lime, and the dome is of 1:2:4 concrete.

The floor is reinforced with 3-in. No. 10 standard expanded metal. The floor and walls are tied together by 48 equally spaced 1-in. square twisted rods 11 ft. long, bent 90° to a 10-in. radius, with a 3-ft. leg vertical. The wall reinforcement is of round rods varying from 1½ in. diameter at the bottom to ⅝ in. at the top. These are properly spaced and carried in the forms by 12 equally spaced vertical frames made of ⅜x 1½-in. steel bars.

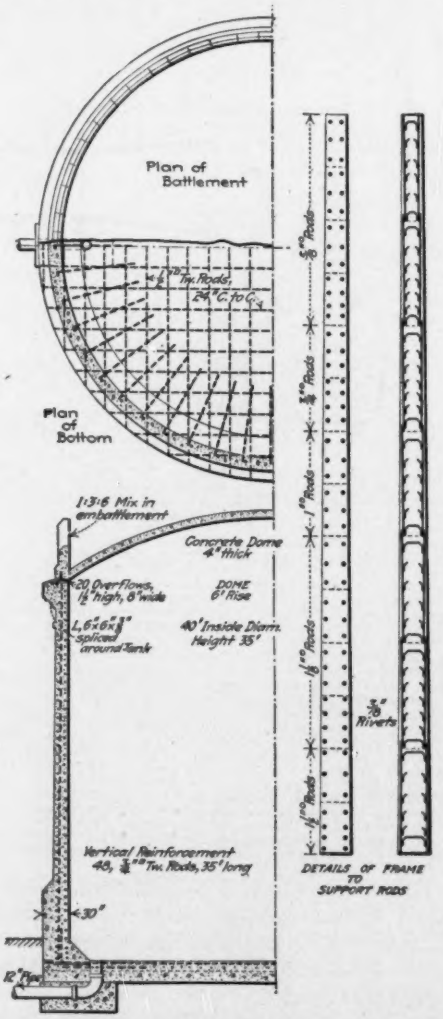


FIG. 2. STRUCTURAL DETAILS OF THE DUXBURY, MASS., REINFORCED-CONCRETE TANK

Each rod frame is built in sections 5 ft. 2½ in. long bolted to the next section by two ½-in. bolts in each leg. The brackets and rod hangers, or rests, are riveted to the legs by ⅜-in. rivets, two on each leg. One frame complete consists of seven sections, the upper fitting into the lower, telescopic fashion, so that the space between the leg plates at the bottom is 13½ in. and at the top 9 in.

In the dome is a 2-ft. covered trap door not shown in the illustration. The battlements were an afterthought and are precast blocks keyed into the rest of the structure by ¾-in. short rods, or dowels.

In order to prevent water from passing through joints, thin steel bands 4 in. in width were inserted so that one-half the width was imbedded in the old work and one-half in the new.

The contract for building the tank was let in August, 1914, to the R. L. Whipple Co., Worcester, for $7115. The battlements cost about $1000 more. Wm. S. Johnson, of Boston, Mass., was designing and consulting engineer, and Harold F. Kellogg, of Boston, was the architect who is responsible for the battlements. The tank was built for the Duxbury Fire and Water District, which includes summer colonies on the South Shore, near Plymouth.

The Second Rebuilding of the Water-Works of Montrose

By P. W. Pinkerton*

The City of Montrose, Colo., thanks largely to an alkaline soil, is about to rebuild its water-works system the second time in 27 years. The first plant was built of kalamein pipe in 1888. The second, costing $100,000, was laid with wood-stave pipe in 1905 and has already failed, although still in use in a crippled condition. Only $10,000 of a total of $120,000 in bonds has been paid, the profit from the operation of the plant always having been "needed for something else." Of the $10,000 of bonds paid off, $9000 was saved during the first year of city-manager government, and there is no reason why this saving should not be repeated annually.

The first supply was obtained by pumping from the Uncompahgre River, but the development of the cyanide process of treating ores and its use in the mills of Ouray County so contaminated this source that when the distribution system began to fail and was also outgrown another supply was looked for. The one finally chosen was the Cimarron Canal, an irrigation ditch by which water from the Cimarron River is carried 26 miles and thence through a long riveted-steel inverted siphon to the Montrose Reservoir. This reservoir, which is the only large storage on the system, is peculiar in that it is located in a natural site almost squarely on the top of a pronounced divide. If there had not been this natural site, in a place where it would be least expected, the storage problem would have been a serious one, as water can be used from the canal only 4½ months of the year, ice prohibiting the running of water between Nov. 15 and May 1, and the May and June water being too muddy for use.

The present capacity of this reservoir is 85,000,000 gal., which is barely sufficient to supply the 3300 people of Montrose during the months when there is no additional supply. It is 14 miles from the city, at El. 8000 ft. above sea level. The water is taken from it for 10 miles through an 8-in. wood-stave flow-line pipe to a small concrete reservoir at El. 6161 ft. The line between these two reservoirs was originally built with a sort of check valve, which in the plans was called a "pressure reducer," located at the end of every 100 ft. of vertical fall. The effect of these valves was to keep the whole line under pressure all the time, and they had to be removed. As a flow line the wood pipe has stood up satisfactorily.

The concrete reservoir holds only 1,900,000 gal., which is about a three days' supply for the town during the irrigating season. Pressure is not obtained directly from the concrete reservoir, but from a wooden tank holding 100,000 gal., two miles from the city limits, at El. 6009 ft. This gives 95 lb. pressure in the heart of town. The line between the concrete reservoir and the tank is of 8-in. diameter and from the tank to the city, 10-in.

Montrose is situated in one of the worst alkali spots in western Colorado. Nearly all the bottom land in the agricultural district surrounding the town had to be drained before it would yield the marvelous crops that have made the Uncompahgre Valley famous. It is this alkali which has already destroyed two water systems and made the community afraid of anything that has to go into the ground, whether it be pipe or building foundation.

Fairness to the wood-stave pipe compels the statement that the pipe was miserably laid. After being delivered it was placed in uncovered piles or left scattered along the line through a whole summer's heat. In the hauling no care was taken to prevent its being rubbed by chains or wheels, and it was very roughly handled. When laid it took parts of it over a vear to soak up sufficiently to carry water.

But this does not explain the three ways in which the pipe has finally failed; namely, the complete corrosion of the wires, the rotting of the pipe and the rotting of the hubs. These occur separately, there being places where the hubs have rotted off although the wires and the rest of the pipe are still good. This seems to be due to a species of dry rot, the hubs not being saturated as is the rest of the pipe. It occurs anywhere on the system, although on the flow line it does no damage. The rotting of the pipe itself occurs largely on the flow line, where again it has done no damage, as the pipe still maintains the conduit shape and carries the water where there is no pressure.

The chief source of difficulty is in the corrosion of the bands, which are often so completely gone that they are hardly visible on the pipe. In repairing broken wires, clamps costing 75c. each are used, made of two pieces of 1½x⅜-in. flat iron in a semicircular shape with flanged ends, the ends being bolted together on the pipe. Care is taken to alternate the location of the pairs of bolts, one clamp being bolted at top and bottom and the next at the sides. Otherwise, the tightening of the bolts would ruin the shape of the pipe. Where both conditions exist—the pipe rotten and the wires gone—there is nothing to do but replace, as the clamps will cut right through the rotten pipe. A broken wire will require from 7 to 22 clamps to replace it. Montrose has city blocks that are clamped solidly from one end to the other, but occasionally leaks appear which compel the uncovering of the line and the tightening of the bolts.

*City Manager, Montrose, Colo.

The unit-cost system installed by the city manager was in shape to give accurate data after May 1, 1914. Since that time the breaks have been at the rate of 262 per annum. The total expense of repairing these was at the rate of $2960 per year, or $11.30 per break. Capitalized at 5%, this represents an investment of $59,200. In figuring on the replacement of the system, however, there are two other things to be taken into account—the inconvenience to consumers in having the water turned off for from two to four hours every 1⅔ days, and the lack of fire protection. In June there were 27 breaks, which is an average of one every 1 1/10 days. During July there were 18 breaks, an average of one every 1 7/10 days. Five of these occurred in two days, July 27 and 28, making it almost impossible to keep the town supplied with water during those days and entirely impossible for a period of seven hours during the night of the second day, in spite of the fact that a full crew was working constantly.

When it was decided to rebuild the system the city manager, although himself a civil engineer, recommended the employment of experts. The city council engaged Burns & McDonnell, consulting hydraulic engineers, of Kansas City, Mo., to investigate all phases of the city's needs. The report of the engineers considers seven possible sources of supply, only two of which will be of interest here. These are (1) a reinforcement of the present source, and (2) the Gunnison River, through the famous Gunnison Tunnel of the United States Reclamation Service.

To continue the present source it would be necessary to raise by 10 ft. the embankment of the Cerro Summit Reservoir, thus increasing the storage capacity from 85,000,000 gal. to 160,000,000, the water to be stored when its quality is good. To utilize the Gunnison River supply, it would be necessary to build (1) a pipe line or flume through the tunnel for use when water is shut out of the tunnel during the winter and for tunnel repairs, and (2) also to build a connection from the Gunnison Canal to the city pipe line. The details of this and some minor work will not be gone into here, as it has been decided that for the present year no work will be done except to replace a siphon, rebuild the supply main from the small concrete reservoir to the town with 13,650 ft. of 12-in. and 7600 ft. of 10-in. cast-iron or steel pipe and replace the distribution system, which work will require 1600 ft. of 10-in., 2220 ft. of 8-in., 12,500 ft. of 6-in., and 22,630 ft. of 4-in. cast-iron or steel pipe, besides valves, valve boxes and fire hydrants, all estimated to cost $70,000.

The specifications provide that if steel pipe is used:

All pipe and specials shall be coated inside and outside with a coal-tar varnish or similar material, and furthermore, the pipe shall be furnished with what is known to the trade as "National Coating" or its equal in quality. The contractor's attention is again called to the condition of soil and character of service to which the pipe will be subjected. The water-supply may at times be slightly alkaline. The soil in which the pipe will be laid contains a considerable amount of alkali and at one place for approximately 800 ft. it will pass through an alkali sink, which is very strongly saturated with alkali and which is always wet. Especial care should be taken in the coating to meet these conditions. The pipe will be laid with an average 3-ft. cover of earth as there will be little trouble from heat.

The writer has made some experiments to determine the value of this coating as a protection against alkali, but these experiments are not yet completed nor will they be until about the time of the opening of bids on May 6. He believes, however, that chemically the pitch is not affected by an alkaline condition similar to that existing at Montrose. The one further question which has to be determined is whether or not the mechanical process of applying this coating by wrapping the pipe with thoroughly pitch-saturated burlap is a sufficient protection.

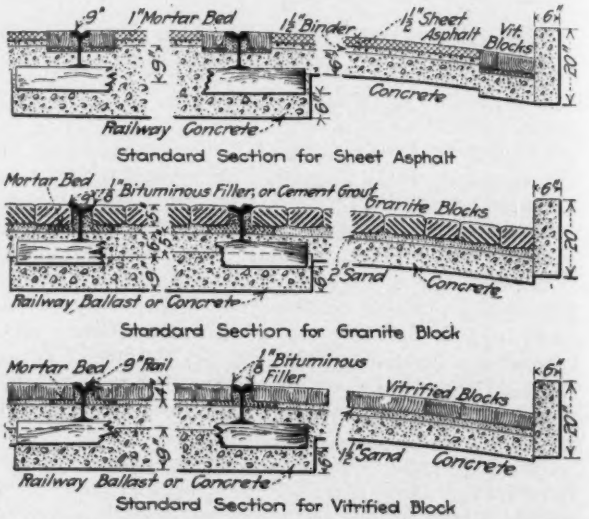
It may be added that the present distribution system consists of mains laid in one direction only (parallel to the sides of the lots), requiring service pipes the lengths of whole blocks, or more, along the lot fronts or through alleys.

Baltimore Experience in Paving Street-Railway Tracks

By Harry D. Williar, Jr.*

Baltimore, Md., a few years ago was known as the "City of Cobblestones"; today it enjoys the distinction of having more new pavements than almost any other city. Yesterday in the eyes of the paving engineer it stood for naught; today it is pointed to as a model.

One of the most difficult problems in this repaving work was the important question of how to pave the railway area. It is well known that most of the failures in pavements occur on streets with car tracks, and that about 50



Figs. 1-3. Cross-Sections of Standard Pavements, Baltimore, Md.

per cent. of these defects are adjacent to and run parallel with the rails.

Baltimore Practice

In Baltimore the procedure of construction in the railway area of a street is as follows: The paving contractor grades to the subgrade of the pavement (usually about the top of the ties). The railway company grades to a sufficient depth to allow 6 in. of ballast under the ties after the rails have been brought to the established grade, ballasts the track, welds the joints and fully prepares the roadbed to receive the pavement. The paving contractor then lays the concrete base and wearing surface from curb to curb, on which he gives a 5-yr. guarantee. After the expiration of this time the city is responsible for the

*Assistant Engineer, Baltimore Paving Commission, Baltimore, Md.

maintenance of the pavement, except the portion between and 2 ft. on either side of the outer rails (the railway area), for which the railway company is responsible.

GRAVEL BALLAST—The paving materials used in the railway area are usually vitrified brick, granite block, scoria and wood blocks and some little sheet asphalt with wood-block liners (Figs. 1, 2 and 3). It has been found that, although these materials were laid with great care and under close inspection, on some of the pavements the parts adjacent to the rails are not holding up. These defects became evident shortly after the completion of the pavements.

An examination showed that the damage done the pavement was due to defective street-railway track construction. Gravel ballast was used by the railway company, and as a result the rails at frequent intervals had an up-and-down motion, against which no pavement, no matter how well constructed, could stand. The use of gravel ballast was ordered discontinued, and crushed stone was used in its place.

STONE BALLAST—After the substitution of crushed stone for gravel ballast was made, paving failures in the vicinity of the rails became far less frequent, but even then were not entirely obliterated. Now and then, where the tamping of the stone ballast was not thorough, the rails would start to "work"; and in a short time the paving blocks adjacent to the rails would begin to loosen and, in the case of sheet asphalt, disintegration would start.

It was thought that these failures were due to two reasons: (1) The tamping of the ballast was not made sufficiently solid, and (2) the cement grout was not allowed to get its proper set before it was disturbed by the passing of heavy cars.

The first cause was rectified somewhat by closer inspection of the tamping on the part of the railway company; and to keep the jar from the pavement while the cement grout was setting an attempt was made to have the railway company install temporary crossovers at frequent intervals where paving work was in progress, to avoid the track adjacent to the newly grouted blocks. This plan met with much opposition from the railway company, however, and in the end proved impracticable. It was discontinued for two reasons: (1) Owing to the schedule of the cars, crossovers could be put only at 400-ft. intervals, and (2) as 400 ft. is hardly a day's run for the paving contractor, it meant continuous disagreement between him and the railway company over hold-ups and delays. When the contractor was ready to grout, the railway company was not there with crossovers, and *vice versa*.

In place of diverting car traffic to keep the jar from the newly grouted pavement, it was decided to tack to the wood fillers of the rail web a ⅛-in. strip of felt or a bituminous material from 4 to 6 in. deep on each side of each rail (Figs. 2 and 3). This "shock absorber," coming between the rails and the pavement, proved very satisfactory, for it relieved the pavement of any direct vibration and thus gave the cement grout sufficient time to get its proper set. Baltimore is now using this last-mentioned form of construction on all car-track streets except those in the heart of the business district. The results thus obtained are very satisfactory.

CONCRETE BALLAST—In the congested downtown section of the city, on such thoroughfares as Baltimore, Howard, Eutaw and South streets, where both vehicle and car traffic is very heavy, the railway ties are laid on concrete.

The following is the method used by the railway company in placing concrete under the ties, and it is strongly advocated by the writer, for in Baltimore it has practically solved the problem of preventing paving failures in the railway area.

The roadbed is prepared in the usual way, the stone ballast being spread and tamped to a depth of 6 in. under and 3 in. up from the bottom of the ties. After the rails are brought to the proper grade and line and the whole construction is "tight" the concreting begins. This consists in flooding the entire stone ballast with a 1:2 mixture of cement grout of about the consistency of cream.

This operation is readily done, without interruption to car traffic, by the use of a small continuous mixer (the Coltrin concrete mixer is recommended) placed just outside the railway area, with chutes for conveying the grout to the ballast (Fig. 4). This grout should be applied

FIG. 4. GROUTING BROKEN-STONE BALLAST TO FORM CONCRETE, BALTIMORE, MD.

to one place until it flushes to the surface of the stone. To obtain such results it is found that one bag of cement is consumed for about every running foot of single track. The total extra cost (over plain broken stone ballast), including labor, material and wear on mixer, is estimated at 52c. per lin.ft. of single track.

One of the principal points gained by this form of construction is that it shows up very clearly every weak place during the progress of work. All loose or poorly tamped ties are made apparent by the bubbling and oozing up of the grout as a car passes over. Failures in the finished pavement have been avoided by immediately retamping such ties, which in many cases would otherwise have been overlooked.

It has been found that this grout, when properly applied, will penetrate the ballast to the subgrade (Figs. 1, 2 and 3). It forms an excellent concrete and assures a solid track foundation, free from vibration, upon which the life of any pavement in the railway area depends.

R. Keith Compton is Chairman and Consulting Engineer of the Baltimore Pavement Commission and R. M. Cooksey is Principal Assistant Engineer, to both of whom credit should be given.

An Unusual Public-Utility Case has recently been decided by the Indiana Public Service Commission. Petitions of the cities of Tipton and Noblesville asking to have the rates of the Indiana Gaslight Co. for natural gas increased from 50—30c. for 1000 cu.ft. to 75—30c. were granted. The local supply had failed, and the necessity of bringing gas from a distance made it impossible to continue service at the old rates.

Concrete Viaduct of a New Type; Eastern Viaduct, New York Connecting R.R.

The New York Connecting R.R. (Pennsylvania-New Haven) includes about a mile of retained embankment that represents a radically new type of structure originated by the Consulting Engineer, Gustav Lindenthal. The walls alone would not be self-sustaining under earth pressure, but they are held together by steel tie-rods spaced 10 ft. vertically and horizontally. The principal structural service of the concrete walls is to serve as girderless slabs to receive the earth pressure and transmit it to the supporting tie-rods.

This novel type of construction was used because it gave a large saving in cost over the much heavier gravity sections of retaining-wall, and because it was also cheaper than steel viaduct—piers and plate-girders—for which the economical height was above 65 ft. in this case.

Reducing the Earth Pressure

The structure at its greatest height is 65 ft. from ground to track level. Ordinary earth-pressure conditions would require very heavy tie-rods to hold so high a fill. Economy dictated the use of all available means for reducing the earth pressure without abandoning the idea of solid fill. Two novel means were employed for the purpose: (1) Ramming the earth fill in place—this was done by pneumatic rammers, as described in the issue of Nov. 5, 1914, p. 938; (2) provision of ample drainage to keep the fill thoroughly dry from top to bottom.

With the fill consolidated by ramming in thin layers and the assurance against moisture disturbance which the system of drainage gave, a remarkably steep slope of repose of the fill material could be figured on. The engineer's judgment placed this slope at about 3 on 1; more precisely, the plane of rupture was taken as having a steepness of 6 on 1 (see abstract of calculations, below).

No Expansion Joints

The walls are continuous, no expansion joints having been provided. Horizontal reinforcement rods amounting to about ½% of the concrete section were found to be sufficient to resist all temperature stresses.

The walls stood two winters without the fill in place (exposed to maximum temperature variations) and one winter since the filling and have suffered no injury.

The prime reason for avoiding expansion joints was the view that the freezing of water in them may produce stresses just as objectionable as those which the joints are designed to prevent. Mr. Lindenthal states it as a broad principle that

> **Expansion joints should in my opinion be avoided whenever possible, since water gets into them which, in freezing, exerts pressure in the joints like a wedge.**

Description

Fig. 1 gives typical drawings of the viaduct. The two side walls are connected by transverse walls about 50 ft. apart and have, therefore, ample stability to resist wind pressure before the fill is in.

The tie-rods, which resist the earth pressure, are 2¼ in. in diameter and are spaced 10 ft. apart, vertically and horizontally. They are anchored into the concrete by longitudinal and vertical 8-in. 16¼-lb. channels, which also add bending strength to the wall slab. While the earth filling was being placed these rods were surrounded with concrete to form an 8-in. square section for protection against rusting.

In every tie-rod section of the structure there is built against each wall a dry-rubble chimney, or drainage well, 4x4 ft. in plan, extending up from a large weeper near the base to just below the top of the wall. Thus, there are two of these chimneys for each 10-ft. length of viaduct.

The earth fill was taken from Sunnyside Yard of the Long Island R.R., at the southerly end of the viaduct. In place it stood up with almost vertical face (of excavation); this is a most important quality of the filling

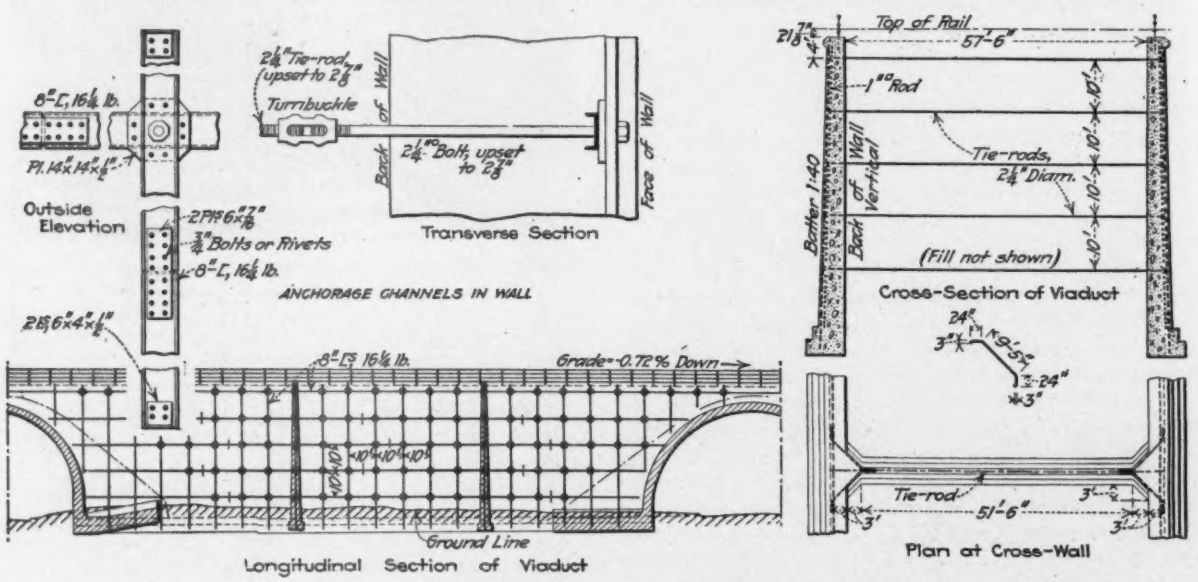


Fig. 1. Details of Eastern Viaduct, New York Connecting R.R.

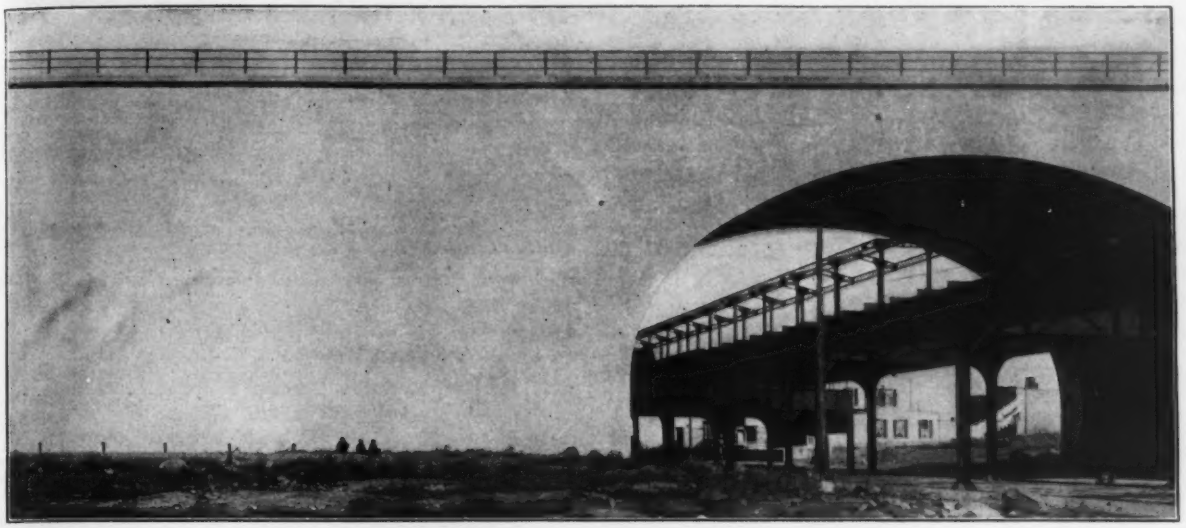


FIG. 2. EARTH-FILLED CONCRETE VIADUCT WITH TIE-RODS

(New York Connecting R.R., Eastern Viaduct at Debevoise Ave. crossing. Under arch is an uncompleted station of the new Queens elevated railway)

material in this case. It consisted of a mixture of sand and clay just cohesive enough to pack hard under pressure or ramming—a natural puddle. It was dumped from temporary tracks on the top of the viaduct and spread to a 12-in. layer, which was then rammed with pneumatic foundry-rammers.

INVESTIGATION OF VIADUCT WALLS

(Abstract)

Plane of rupture BC, Fig. 3, assumed as slope of 6 on 1. Surcharge due to four tracks of E60 railway load plus 100% impact, 840 lb. per sq.ft., equivalent to 7-ft. depth of earth.

Weight of earth prism ABC, 36,000 lb. per lin.ft.; surcharge, 8400 lb.; total weight, 44,400 lb. Angle of friction on plane of rupture assumed at 45°. Friction on back of wall neglected. Earth pressure found equal to 30,000 lb. per lin.ft., or 300,000 lb. per 10-ft. length of wall.

METHOD A; NO BASE FRICTION—If prism ABC is assumed to act as a wedge, the horizontal pressure on the wall would be uniform for the whole height of the wall. The tension in each rod would be $300{,}000 \div 6 = 50{,}000$ lb.

The usual assumption is that the pressure acts at the center of gravity of the wedge, which is taken as being 22.5 ft. above ground, or 7.5 ft. from the center of gravity of the six rods (Fig. 4). The eccentric application of the earth thrust would produce a moment of $300{,}000 \times 7.52 = 2{,}250{,}000$ ft.-lb., giving a stress in the extreme rods of 1280 times the distance of outer rod from center of gravity of rods, or 32,000 lb.

The resultant stresses are shown at the left in Fig. 4. The stress on the bottom rod is 82,000 lb., which, with 4 sq.in. of area, gives 20,500 lb. per sq.in.

METHOD B; WITH FRICTION ON BASE—Considering friction of wall against sliding, the distribution of the rod tensions is somewhat different from that given above. The wall is reinforced against bending by 8-in. channels spaced 10 ft. center to center, longitudinally and vertically, and intersecting at the tie-rod connections. Assuming a frictional force against sliding of 60,000 lb. per 10-ft. length of wall, the average compression it produces is 10,000 lb. per tie-rod, and owing to its eccentric application it causes a bending moment of $60{,}000 \times 40 = 2{,}400{,}000$ ft.-lb. and a stress of 1370 lb. times distance of extreme rod from center of gravity.

The resulting stresses due to the frictional force and the total stresses including the effect of friction are shown in the two right-hand columns, Fig. 4. They show nearly uniform stress in all rods (maximum of 10,500 lb. per sq.in.)

As it is not probable that the tension stress in the bottom rod will be smaller than that in the top rod, the assumption made for the frictional force seems to be reasonable.

That part of the earth thrust counteracted by friction at the base of the wall is assumed to be transmitted by bending in the wall. The maximum amount of this bending is 1,050,000 ft.-lb. per 10-ft. section 40 ft. below the upper rod (see horizontal section, Fig. 5); it causes a concrete compression of 490 lb. per sq.in. and a steel tension of 65,000 lb. per sq.in. The steel tension, as computed, is much higher than the actual because the presence of the tie-rods will prevent this bending from developing.

The true stresses will be somewhere between those obtained by method A (neglecting friction on base of wall) and method B (considering friction). All dimensions appear safe in this case for either of the two extreme assumptions.

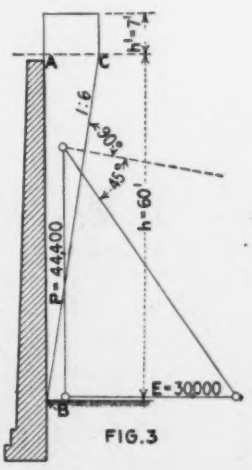


FIG. 3

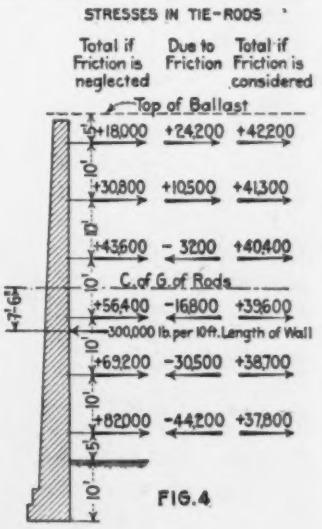


FIG. 4

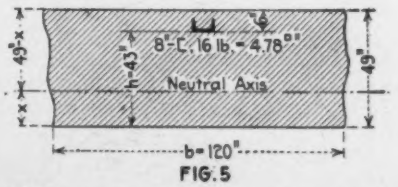


FIG. 5

If the filling were sand and gravel, which have a flatter angle of repose in their natural condition than above assumed and are more easily disturbed by loads, then larger lateral stresses and heavier sections in the walls and tie-rods would have resulted. The dimensions of any retaining-wall depend, of course, upon the kind of filling behind it, which may range from water to rock. In the end the safety of the structure depends upon the skill and judgment of the engineer who makes the assumptions.

Practice in Paving Street-Railway Tracks

The practice of putting concrete under the ties has been adopted in a number of cities. In some instances the concrete is under and around the ties, but only of the usual thickness between the ties. The best practice, however, seems to be that adopted in Baltimore (which is described

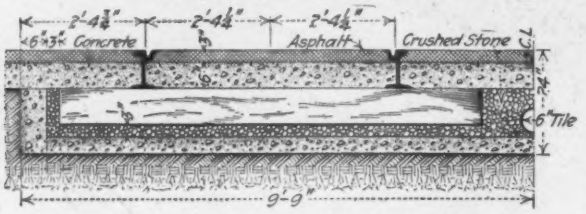


FIG. 1. STANDARD PRACTICE IN HARTFORD, CONN.

on p. 884 of this issue) of having a base of uniform thickness extending to 6 in. below the bottoms of the ties throughout the track area.

This form of construction is generally objected to by the railway companies because of the rigidity of the track foundation and because of the difficulties in making rail and tie renewals. With a modern first-class grouted granite-block pavement the life of the pavement is probably two or three times the life of the rails, and possibly the ties. The rail-renewal problem has been partly solved by a Chicago experiment in the use of a composite rail, the wearing part of which can readily be replaced (see ENGINEERING NEWS, Sept. 28, 1911); and white-oak ties incased in concrete may of course last as long as the pavement; but as yet this form of construction has not been extensive enough to prove this.

A form of track construction not open to either of these objections is shown in Fig. 1, which illustrates the standard method of construction in Hartford, Conn. This method has been in use there a number of years and has proved thoroughly satisfactory. Using this form of construction, it has been possible to lay sheet asphalt against the rail without the use of block liners; and in practically every instance the result has been all that could be desired, as the illustration (Fig. 2) shows. This is a view in Asylum St., a narrow, heavy-traffic street, which had been paved as illustrated for several years.

A common but generally unsatisfactory form of track construction for sheet asphalt and bituminous pavements is shown in Fig. 3. This method is much used in the smaller cities and on village streets having one of the various forms of bituminous pavements.

The track foundations designed by the Committee on Way Matters of the American Electric Railway Engineering Association require 6 in. of concrete under the ties in "heavy water-retaining soils and uncertain made-ground, with the heaviest traffic and cars." (See ENGINEERING NEWS, Nov. 26, 1914, p. 1079, and Dec. 17, 1914, p. 1222, for illustrations.)

FIG. 2. ASYLUM ST., HARTFORD, CONN., SHOWING SHEET ASPHALT AGAINST THE RAIL

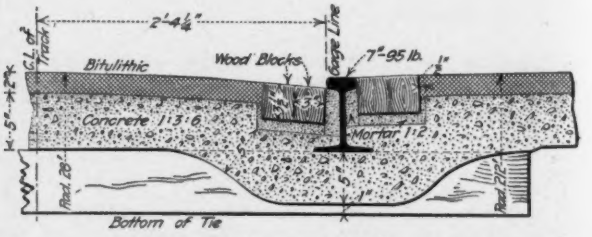


FIG. 3. COMMON METHOD OF TRACK FOUNDATION ON BITUMINOUS PAVED STREETS

The Cost of Hauling Material for road construction is discussed by Fred Tarrant, Division Engineer of the Illinois Highway Department, in that department's official publication for April. He estimates the cost of hauling road material over country roads with teams at 40c. per cu.yd. for the first mile and 20c. for each additional mile. In building a waterbound macadam road in Jersey County, Ill., teams hauling stone an average distance of 1½ mi. showed a cost of 33c. per ton-mile. On the same road a steam traction engine hauling four 7-yd. wagons an average distance of 3 miles showed a cost per ton-mile of 15c. Neither of these figures included interest and depreciation on the equipment used. In building 12½ mi. of waterbound macadam road, an industrial railway outfit was rented from Orenstein-Arthur Koppel Co., of Koppel, Penn. The outfit consisted of two 20-hp. locomotives, six miles of portable track and 60 cars. The road to be built was practically level, except three hills on which the grades were about 4%. Using this industrial railway with rented equipment, including the freight both ways on the outfit from Koppel, Penn., the total cost of hauling was 14.8c. per ton-mile. Mr. Tarrant estimates that the cost of team hauling on the same work would have been 28 or 30c. per ton-mile, provided the teams could have been obtained. Had the township owned the industrial railway, the cost of hauling with it could have been reduced to 11c. per ton-mile.

An Electrically Operated Hydraulic Dredge*

Hydraulic dredges operated by electric power have been known for several years, so that the electric hydraulic dredge recently built by the Norbom Engineering Co. for extensive excavation and filling-in work around Lake Nakomis, Minneapolis, is not a distinct departure. Electrically operated dredges, however, are interesting enough in themselves to deserve description. Furthermore, electrical engineers will probably be surprised to learn that the pump on this dredge was operated by a synchronous motor. The manufacturer of the dredge believes that the induction type of motor is best for this class of service, but the owners of this dredge, the Northern Dredge and Dock Co., agreed to take the risk of the synchronous motor in view of the economical operation and the lower rate per kw.-hr. obtained by its use.

New Electric Hydraulic Dredge†

The dredge in use at Minneapolis has a comparatively large pump runner, being 54 in. in diameter. This insures an efficient working length of the vanes. It is driven at two speeds—300 and 200 r.p.m.—by a 500-hp. synchronous motor running at 720 r.p.m. Speed reduction and speed change are effected through two sets of cut helical gears running fully inclosed. A lever on top of the gear box, connected with a positive clutch, locks the gears in position for one speed or the other.

With the synchronous motor the power is ample for the work intended. The torque at full speed, with the motor running as an induction motor, is about 30% of the full-load torque. There is, therefore, no difficulty in starting the motor as an induction motor and switching on the direct current as it comes up to speed. Two speeds are sufficient; it is intended to run on the lower speed with pipe lines up to 1500 ft., at about 10- to 12-ft. lift, while for more than this the higher speed is required. Under average conditions the dredge excavated 5600 yd. per day, while the daily maximum exceeded 7000 yd. in 24 hr.

The cutter machinery is operated by a 50-hp. slipring-type motor. The operating and winding machinery is driven by a 30-hp. motor of the same type, and consists of five drums with friction clutches and brakes, and two winch heads. A water-jet exhauster, operated from an auxiliary pump, is used for priming the main pump.

The current used is 2300-volt alternating, three-phase, 50-cycle, and is used without transformation on the three principal motors. For the service-pump motor, the voltage is stepped down. A motor-generator set furnishes energy to a searchlight, an electric horn and the regular ship lighting. The hull of the dredge is 80 ft. long by 24 ft. wide and 7 ft. deep, and the average draft is 40 in.

*An electric dredge whose pump was operated by an induction motor was described in "Engineering News," Aug. 28, 1913, p. 432.

†Information contained in this article was furnished by the Norbom Engineering Co., Philadelphia, Penn.

Cost of Cleaning Various Kinds of City Pavements*

The Philadelphia Bureau of Highways has made some special tests to determine the comparative costs of cleaning city pavements by machine brooms. The average test results compare quite closely with actual average costs for work throughout the city, so that these results are valuable as a matter of record, as well as to throw light on a subject which has never been very extensively studied.

The table is almost self-explanatory. The costs given do not include the cost of water, which may be computed in this instance at 4c. per 1000 gal., or 0.4c. per 1000 sq.yd. The costs do include the removal of the sweepings, averaging 0.23 cu.yd. per 1000 sq.yd. (about 8.7c. per 1000 sq.yd.). They include operation and direct supervision only, not overhead, depreciation and other charges. The following are the rates of pay: Foremen, $2.50 per day; gangmen, $1.75; machine-broom hire, $5.50; dirt wagon, $5; dirt cart, $3.50; sprinkler, $5. The work in the different districts is generally done by different contractors, so that the varying costs are due to varying conditions such as would be met with in comparing cost data in different cities.

STREET-CLEANING COST DATA ON MACHINE-BROOM WORK; PHILADELPHIA (PER 1000 SQ.YD.)

	Computed from Special Test Data (in Cents)																From District Daily Reports		
	Granite Block				Brick				Wood Block				Asphalt				Average Unit	Average Unit	
	Traffic			Unit	Traffic			Unit	Traffic			Unit	Traffic			Unit	Cost All	Cost All	
District	Heavy	Medium	Light	Cost	Heavy	Medium	Light	Cost	Heavy	Medium	Light	Cost	Heavy	Medium	Light	Cost	Classes	Classes	District
1-A	33.7	18.3	20.0	20.1	....	....	20.8	20.8	....	....	....	....	....	....	19.8	19.8	20.2	20.8	1-A
1-B	....	23.7	....	23.7	....	....	21.8	21.8	....	....	....	....	....	....	14.7	14.7	22.0	19.4	1-B
2	29.0	25.9	....	28.1	....	22.5	....	22.5	....	20.0	....	20.0	....	18.7	....	18.7	26.2	29.0	2
3	25.5	22.0	....	23.8	....	....	....	....	21.7	....	....	21.7	....	....	....	....	23.3	30.2	3
4-A	28.2	25.0	20.0	25.4	....	22.6	....	22.6	....	26.4	....	26.4	....	....	....	....	25.3	25.1	4-A
4-B	28.3	23.1	20.9	26.1	....	22.1	....	22.1	....	....	....	....	....	24.3	17.8	18.6	23.6	22.6	4-B
5	30.5	22.7	20.7	25.0	....	23.1	....	23.1	....	....	....	....	....	19.4	....	19.4	22.8	33.6	5
6	....	25.9	23.3	25.1	....	27.7	17.8	21.0	....	....	....	....	....	....	....	....	23.9	23.2	6
Average unit cost				24.5	....	....	....	21.7	....	....	....	21.7	....	....	....	18.7	23.2	26.9	...

The Oxford Pike Service Test Concrete Road of the Philadelphia Bureau of Highways has a total length of a little over 2 mi. and consists of five principal sections 2250 ft. in length each, differing in composition of the concrete; and each section is subdivided into two smaller sections of 1050 ft. and 1200 ft. each. The 1200-ft. sections are again divided into four sections to provide for experiments with different types of bituminous wearing surface. These sections are surface treated with refined coal tar (Tarvia), refined water-gas tar (Ugite), asphalt cut-back and Unionite (a mixture of rock asphalt and stone grit). The 1050-ft. sections are left without any surface treatment. Transverse expansion joints are placed every 30 ft. Two types of joints are used—armored and prepared bituminous.

The composition of the five concrete sections is as follows:

1st Section	1 part cement, 2 parts sand, 3 parts gravel.
2nd Section	1 part cement, 2 parts sand, 3 parts crushed trap rock.
3rd Section	1 part cement, 1 part trap rock grit, 1 part sand, 3 parts crushed trap rock.
4th Section	1 part cement, 2 parts trap rock grit, 3 parts crushed hard limestone.
5th Section	1 part cement, 2 parts trap rock grit, 3 parts crushed trap rock.

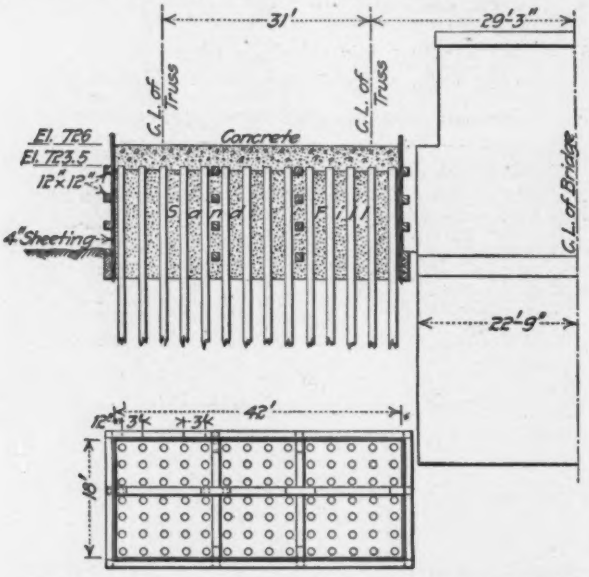
*From information furnished by Wm. H. Connell, Chief of the Bureau of Highways, Philadelphia, Penn.

Field and Office

Sand-Braced Pile Pier

An ingenious form of pier construction was used in the course of rebuilding the Coshocton bridge of the Pennsylvania Lines, which bridge was washed out in the great flood of March, 1913, and was rebuilt a few months ago. This pier is not part of the permanent substructure, but was required to support two heavy double-track spans for a period of several months until they were moved over to permanent position.

The new bridge—three 240-ft. spans—was erected on falsework alongside the old bridge, then swung on tem-



PIER FOR COSHOCTON BRIDGE, PENNSYLVANIA LINES

porary piers and abutments, and traffic was turned over it while the old bridge was torn down, so that the new structure could be rolled over into place. The temporary piers thus were in service under train loads for several months. Substantial construction was required to forestall settlement and all danger from floods.

At the two abutments and one of the intermediate pier points the ground was high enough so that thick concrete mats could be laid directly on the ground and framed timber bents set on them to carry the spans. The other intermediate support, however, was in 20-ft. depth of water, and a pile foundation had to be employed. In view of the great load to be carried, about 4,500,000 lb., stability was believed to necessitate more reliable construction than an ordinary unbraced pile-group would furnish.

The Foundation Co., contractors for the substructure of the permanent bridge and builders of the temporary piers for the new bridge, employed a type of construction devised for the purpose by S. P. Mitchell, president of the Seaboard Construction Co., contractors for the erection of the superstructure. This type is exceedingly simple, but it met the conditions fully and proved satisfactory throughout its service.

Piles were driven 3 ft. on centers both ways to firm bearing, with tops extending above water level. Around the pile group a 4-in. timber box 18x42 ft., braced with 12x12 horizontal timbers and tied together with rods, was constructed, sunk, and jetted 4 ft. into the sand of the river bottom. The box was then filled with sand pumped in. On top of this filling was constructed a concrete cap 4 or 5 ft. thick, embracing and covering the pile heads. Rail reinforcement in this concrete tied it together against the effects of local irregularity of bearing. Framed timber bents carrying the bridge seats were set directly on the concrete cap.

The cost of construction of this temporary pier was not very much greater than the cost of the pile group alone, while thorough stiffening was secured and all danger of damage from high water, floating trash and the like was eliminated.

Monel Metal in Water-Works and Other Engineering Service

The inclusion of the natural alloy, Monel metal, in the study of brasses and bronzes being made by the U. S. Bureau of Standards at the instigation of the New York City Board of Water Supply (whose troubles with defective bronzes were described in ENGINEERING NEWS, Nov. 26, 1914) has aroused renewed interest in the engineering applications of this metal. Monel is a natural nickel-copper alloy (approximately 67% nickel, 28% copper and 5% other metals, like iron, manganese, etc.) produced from ores of the Cobalt, Ont., district. Three of its important mechanical properties are incorrodibility, toughness and retention of strength at high temperatures. Monel metal is produced only by the International Nickel Co., of New York City. Hot-rolled shapes cost upward of 25c. per lb., depending on size and market.

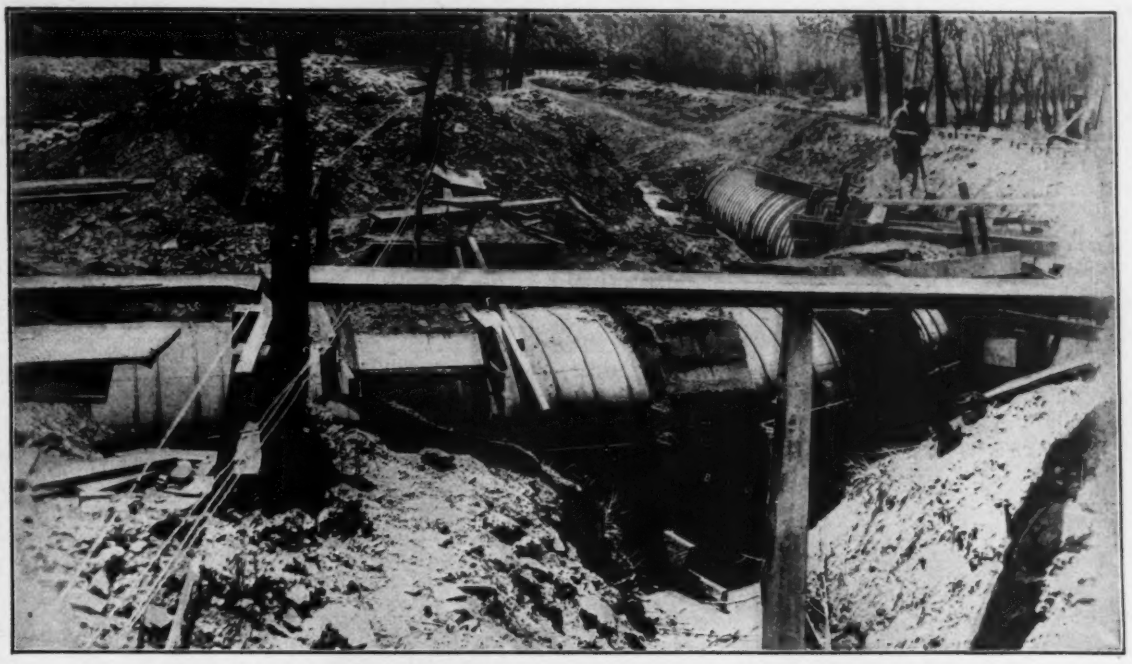
Last year 23,000 lb. of valve-stem forgings were furnished the Kennedy Valve Co., Elmira, N. Y., for the high-pressure fire-service valves of the City of Boston. Monel parts were used in the Boston high-pressure hydrants, made by A. P. Smith & Co., East Orange, N. J. The average of the Boston inspection tests shows:

Tensile strength, lb. per sq.in.	84,300
Yield-point, lb. per sq.in.	56,500
Elongation in 2 in.	38.6%

The alloy is also being used by Wallace & Tiernan, of New York City, for parts of chlorine-dosing apparatus in water-works. The metal is used by the U. S. Navy for valves and valve stems, pump rods and liners, sea-water inlets and outlets and for propellers—wherever the severe service of high temperatures or salt water has to be met. There has been a considerable amount used in the Panama Canal equipment—for cases similar to naval use, for house screens and for transmission-line insulator supports.

A Sharp Curve in a Wood-Stave Pipe

Wood-stave pipe can easily be laid to curves having a radius of 50 diameters of the pipe. When the radius becomes smaller than this, difficulty is encountered in making the turn. The accompanying view shows the method used some years ago in making a sharp curve in a large wood-stave pipe in the Arkansas valley in Colorado. The pipe is under a head of about 15 ft. The curve is composed of five or six sections of pipe, each about 5 ft. in length, fitted end to end and connected by reinforced-concrete collars about 2 ft. wide. This method could be used for making elbows or turns of any angle or curvature in any size wood-stave pipe.—KENNETH A. HERON, Modesto, Calif.



A SHARP CURVE IN WOOD-STAVE PIPE MADE WITH STRAIGHT SECTIONS

Laying Water Main in Tunnel to Avoid Frozen Ground

BY A. COHN*

Laying a water main in a tunnel only a few feet below the ground surface is so unusual as to warrant a brief statement of why and how it was done.

WHY

Gilbert, Minn., a village which had a population of 1500 in 1910, had been getting its water-supply from a mine for five years when, in August, 1914, the mine owners served notice that a change must be made within a year. The village decided to erect a 750,000-gal. filtration plant on the shore of Cedar Island Lake (sometimes called Ely Lake), build a pumping station, lay a 10-in. force main of Matheson-joint steel pipe about 4000 ft. long, and erect a tank on a hill in the village 205 ft. above the lake.

As the village wanted to start the work on Feb. 1, in the dead of winter, bucking frost in open trenching was the chief problem. Frost was found to a depth of 5 ft. below the surface. The line necessarily follows high ground to afford easier digging and because of the irregular contour. The average depth had to be about 12 ft. The first 2000 ft. from the lake was mainly fine sand, which, if open-trenched, would have to be planked throughout. Also, two railroad yards had to be crossed. After much estimating and discussion, it was decided to tunnel the whole 4000 ft.

HOW

On the tangents, shafts 4x5 ft. in plan (inside timbers) were sunk at every 100-ft. station. On curves, a shaft was sunk at the middle and at each end. Each miner drifted 50 ft.

Sets, having a 2½-ft. cap, 5-ft. posts, and 3-ft. mud sills, were used as standards. The sides were lagged up wherever necessary. Grades were given by using a Locke hand level on a 3-ft. stick, with the bottom end of a level rod. Line points were put in each side of the shafts; strings hung from them gave alignment both ways. All drifts holed through well, and lines met satisfactorily.

A windlass at the top of the shaft handled the muck. For the first 20 ft. of drifting, the miner did his own mucking. At 50 ft. he had two and sometimes three muckers, the dirt being handled three times in the drift. The miners averaged about 4½ ft. per day. This was good, considering the cramped position in which they worked. No rock was encountered in the first half, and—working about 112 men—this part was completed in about 2½ weeks. On the last end of the line rocks and hard clay were encountered and blasting had to be resorted to frequently. In all cases 30 per cent. dyna-

*Assistant Engineer, Village of Gilbert, Minn.

mite was used and the ground broke nicely, the amount of extra dirt handled being small.

Forty-three shafts were sunk, the average depth being about 12 ft. About one-half of these were cribbed, using 8-ft. lagging for the top part and then culling the lower pieces to suit. The ends of the lagging rested on the top of the cap, being supported by it.

The Pipe Work

The Matheson-joint steel pipe used for this line was about ⅛ in. thick, was coated with burlap and a tar mixture about $\frac{1}{16}$ in. thick altogether, and had bell-and-spigot joints. After the joints were calked, they were covered, successively, with tar, burlap and tar. On tangents, 20-ft. lengths of pipe weighing about 400 lb. each were used, and on curves, 10-ft. lengths were laid.

Two calkers and two assistants completed the calking in about six working days of 8 hr. each. Shafts at the 1000-ft. stations were made 8 ft. long, to allow the sliding down of the pipe. A little "godevil" was placed under one end of a length of pipe and the pipe snaked to place by six men.

After the pipe was calked the drifts were blasted down and then filled. About 2½ months elapsed from the time the first shovel of dirt was thrown until the last. This method of putting in the pipe cost about one-half as much as open trenching under these conditions. G. A. Arnold of Bisbee, Ariz., had active charge of the work.

Recording the Locations of Water-Main Gate Valves

Failure to find the nearest gate valve on a burst 30-in. water main, until the expiration of 4 hr., is reported to have recently cost the city of New York about $150,000 in damage claims, some of which may result in expensive litigation. This is an extreme case, but it illustrates the urgency of having readily accessible records as well as the necessity of accurately determining the location of all water-main gate valves.

After experimenting with various forms of records, Geo. A. Carpenter, City Engineer of Pawtucket, R. I., has devised and adopted one which answers admirably every need, both of his own and other city departments.

A map of the city is first divided into general sections covering several square miles each, of which the sketch, Fig. 1, shows a full-size portion. The black lines answer for both streets and water mains. The black dots show the approximate locations of all gate valves.

Each of these general or index maps is divided into approximate squares lettered and numbered as is common practice on geographical maps. These division lines are bent when essential to include particular valves or groups of valves which control the flow in mains within the square. These maps are black and white with red cross-section lines. They are 18x10 in. and are folded to be bound in 8x5-in. notebooks.

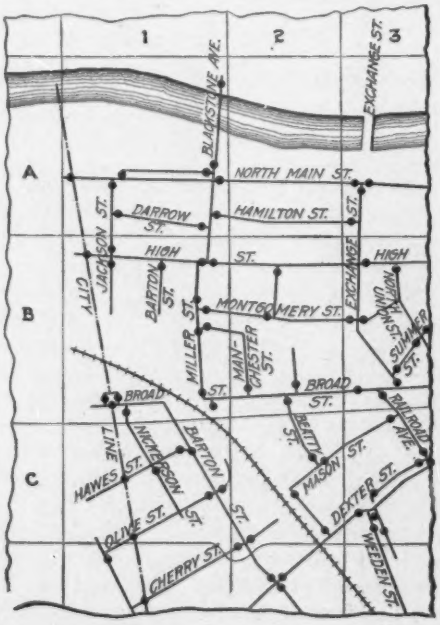


Fig. 1. Portion of Index Map for Recording Valve Locations; Pawtucket, R. I.

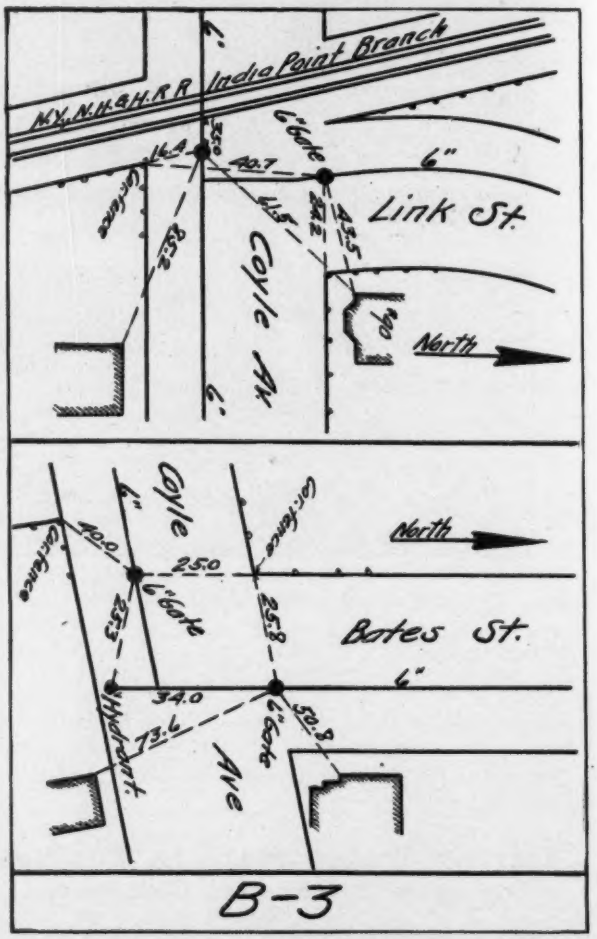


Fig. 2. Page of Valve Record Book; Pawtucket, R. I.

The rest of each notebook is made up of loose leaves like that shown in Fig. 2. These are 8x5 in., in black and white, with sketches on both sides of the leaf. These are freehand sketches; no attempt is made to plot them to scale.

At the bottom of the page is shown a reference mark *B*-3 (this particular example is not a part of general plan Fig. 1). This mark locates the sketch on the general plan

or *vice versa* as the notebook is ordinarily used. The leaves are paged alphabetically.

Three or four of these notebooks cover the entire city and there are copies of each notebook in the various city departments and readily accessible.

If a water main breaks the water department officials have only to take the notebook into the field, note the square, say *B*-3, containing the trouble, and within a few minutes they are able to determine the exact locations of all gate valves in the vicinity.

All valves are referenced by linear offsets to prominent objects, such as street corners, catch-basins, lamp posts, etc.; right-angle offsets are preferred. It is then a simple matter for any laborer to find the valve.

Diagram for Net Section of Riveted Tension Members

The diagram reproduced herewith has been prepared by T. A. Smith, 914 Karpen Bldg., Chicago, on the basis of calculations for true maximum stress, as previously used for a similar purpose of V. H. Cochrane in ENGINEERING NEWS, Apr. 23, 1908. Taking the diagonal section *AB* through rivet holes, Fig. 1, the normal stress and the shear on this section are computed and therefrom the true maximum stress computed by the formula

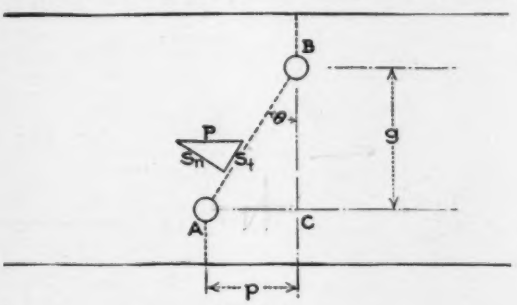


FIG. 1

$$S_m = \tfrac{1}{2} S_n + \tfrac{1}{2} \sqrt{S_n^2 + 4 S_t^2}$$

$$= P \frac{g + \sqrt{g^2 + 4p^2}}{2(g^2 + p^2 - d\sqrt{g^2 + p^2})}$$

The unit-stresses given in Fig. 2, for various staggers in a member having two lines of rivets, show the importance of this calculation. It will be seen that for any stagger less than $3\frac{5}{16}$ in. the diagonal section is weaker than the direct section, although only $1\frac{13}{16}$-in. stagger is required to make the actual net section of metal on the diagonal equal to the net direct section. In other words, the stagger really required is nearly twice as great as that used by some designers.

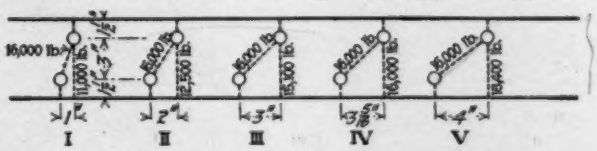


FIG. 2

The above formula for true stress on the diagonal section may be used to compute the amount of rivet-hole deduction which would have to be applied to the diagonal section so that it may be calculated at the same unit-stress as that on the direct section. More than one rivet-hole must be deducted (as proven by the stresses in Fig. 2), but less than two, since two holes would be deducted only when directly opposite each other. The deduction to be made is $1 + x$, where x is a fraction. It can be computed by the formula

$$x = \frac{g}{d} - \frac{2(g^2 + p^2 - d\sqrt{g^2 + p^2})}{d(g + \sqrt{g^2 + 4p^2})}$$

whose value is such that $\frac{P}{g - xd}$ is equal to the allowable working stress.

The diagram gives the values of x for $\frac{7}{8}$-in. rivets.

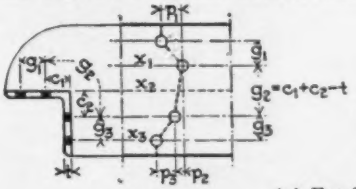
The diagram gives (at $x = 0$) the minimum stagger to make the diagonal section as strong as the direct transverse section; this value may be used in designing.

DIAGRAM FOR NET SECTION OF TENSION MEMBERS

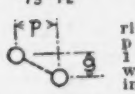
In determining the net section of tension members, the number of rivets n to be deducted may be determined from the formula:

$$n = 1 + x_1 + x_2 + x_3 + \text{etc. where } x = \frac{g}{d} - \frac{2(g^2 + p^2 - d\sqrt{g^2 + p^2})}{d(g + \sqrt{g^2 + 4p^2})}$$

g = gage, and p = pitch or stagger, as shown in the following diagram; d = diam. of rivet hole.

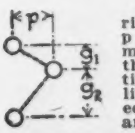


The rivets to be considered in the above formula should be those which give the largest total number of rivets to be deducted whether on adjacent lines or not.



(a) For 2 lines of rivets, the value of p such that n = 1 may be found where gage lines intersect line A-A.

(b) For 3 lines of rivets, the value of p such that n = 2 may be found where gage lines intersect line B-B.



(c) For 3 lines of rivets, the value of p such that n = 2 may be found from the location of a vertical line cutting gage lines g_1 and g_2 at equal distances above and below line B-B.

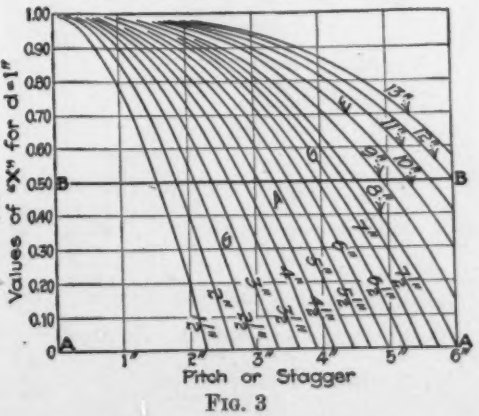


FIG. 3

In case of a completed design or structure the proper value of x is taken for the actual spacing and stagger.

For more than two lines of rivets, the deduction is $1 + x_1 + x_2 + \ldots$, where the x's are the separate deductions for the several divisions between gage-lines. That section must be used which requires the largest deduction, regardless of whether it takes in all the gage-lines or not.

Grouting Equipment for City Tunnel, Catskill Aqueduct

By Walter E. Spear*

The accompanying illustrations show the equipment employed in grouting the City Tunnel of the Catskill Aqueduct (see Engineering News, Feb. 4, 1915). Fig. 1 shows the arrangement of plant commonly used on the Catskill Aqueduct, which was adopted in some sections of the City Tunnel. Air piped from the compressor plant on the surface was delivered directly to the grout tanks at pressures of 80 to 100 lb. per sq.in. for the low-pressure work. When the grouting required higher pressures the air was further compressed to 200 or 300 lb. per sq.in. by means of an auxiliary air-driven compressor or "booster," supplied from the compressor at the surface.

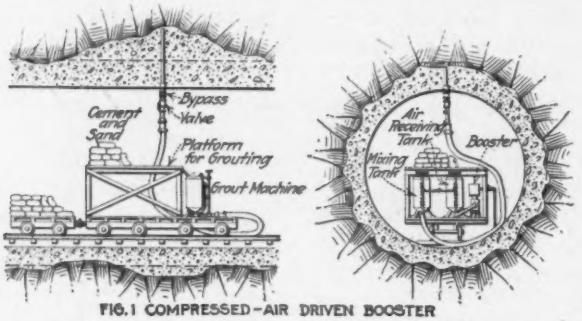


FIG. 1 COMPRESSED-AIR DRIVEN BOOSTER

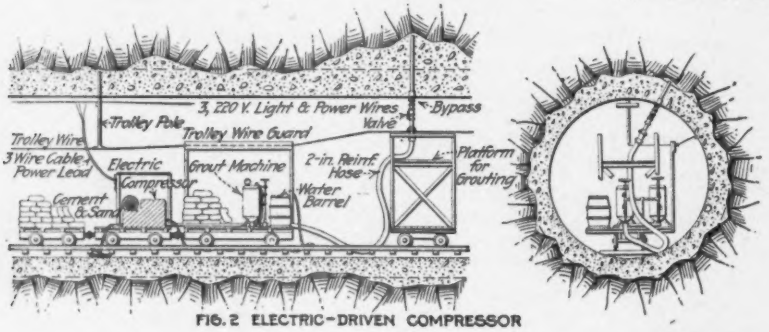


FIG. 2 ELECTRIC-DRIVEN COMPRESSOR

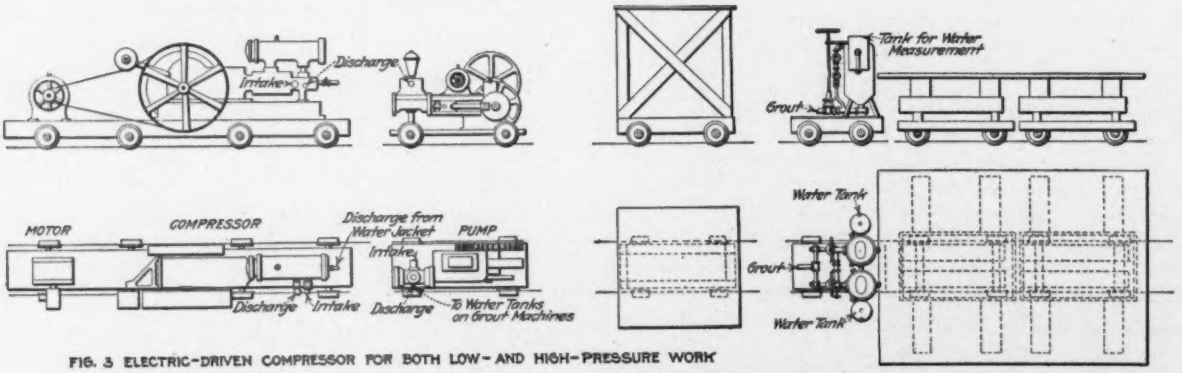


FIG. 3 ELECTRIC-DRIVEN COMPRESSOR FOR BOTH LOW- AND HIGH-PRESSURE WORK

THREE TYPES OF GROUTING EQUIPMENT USED ON THE CITY TUNNEL OF THE CATSKILL AQUEDUCT

(In Fig. 3, a 20-hp. Westinghouse three-phase motor drives a two-stage Ingersoll-Rand belt-driven compressor at 210 r.p.m., delivering 129 cu.ft. of air per min. at a pressure of 350 lb. per sq.in. A 7½-hp. motor of the same type as the larger one drives a double-acting reciprocating geared McGowan pump, delivering 50 gal. of water per min.)

On several sections of the City Tunnel compressed air was furnished for the grouting by small electrically driven compressors mounted on trucks in the tunnel and moved along with the work. Fig. 2 shows a type of equipment in which the low-pressure grouting was carried on as usual with the air furnished from the surface compressor plant, but the high-pressure grouting was done with the small compressor driven from the lighting or power circuits.

Still another style of equipment is shown in Fig. 3. With this plant both the low- and high-pressure grouting was done with the electrically driven compressor in the tunnel. The adoption of this equipment, in which the compressor in the tunnel could be operated only when grouting was actually being done, made unnecessary the more or less continuous operation of the large compressor plant on the surface and effected a further economy in avoiding the piping of compressed air through the tunnel for the grouting work.

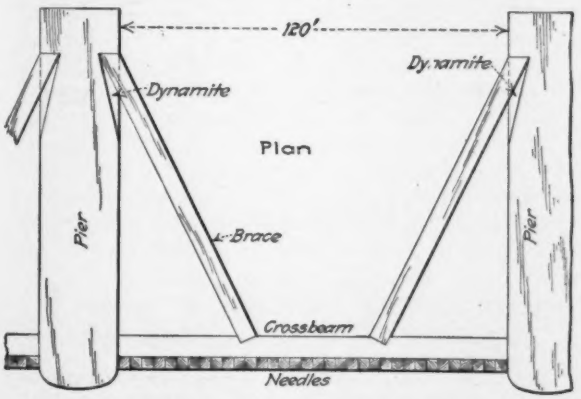
*Department Engineer, City Aqueduct Department, Board of Water Supply of the City of New York, 250 West 54th St., New York City.

Reducing the Sweating of Water-Meter Dial Glasses—O. F. Poetsch, Superintendent of Meters of the Milwaukee Water Works Department, reports a great deal of difficulty experienced by meter readers during certain periods of the year because of the sweating of water-meter dial glasses. Whenever the temperature of the atmosphere outside is lower than that of the water in the meter, the dial fogs, the moisture later condensing into drops, obscuring the hands on the dial, or at least distorting the view of them to such an extent that it is impossible to obtain a correct reading of the meter. It then becomes necessary to heat the glass to remove this moisture or to break it to get a reading. Mr. Poetsch therefore suggests that small holes be drilled in opposite sides of the register box between the glass and the dial to allow a circulation of air; but he adds that while this will remedy the difficulty in a measure, it will by no means entirely remove the trouble. Wherever possible without detriment to the meter or where it will not be tampered with, the dial glass should be omitted, Mr. Poetsch says, and if a covering is necessary to protect the dial face some specially constructed dial cap or register box cover should be designed so as to close automatically. This would offer more protection than a glass and afford the meter reader a means of cleaning the dial if dirty, as well as save all the time now lost in trying to read sweating glasses. Further, it is claimed that there would be a considerable saving of meter repairer's time, since it would no longer be necessary to send men to clean dials or repair broken dial glasses.

A Jam-Proof Dam

The accompanying sketch shows a novel dam on the Wisconsin River at Wausau. This stream is ordinarily steady, but is subject to ice gorges and log jams. The spillway capacity is limited and large steel Taintor gates are used here, as on other Wisconsin and Michigan streams. When logs come down, either from broken jams or booms above, they are apt to pile so high as to overlap the limited clearance of the gate openings. Accordingly, to provide a clean sluiceway from the river-bed up, when needed, a couple of spans of the type illustrated were put in at the deepest portion of the channel.

Piers were put in about 20 ft. apart, with front sills between on the river bottom. A line of horizontal timbers was stretched from pier to pier near the water-surface line, but not in any way secured to the piers except to rest on projections. The sluices were closed with needles, which were supported by the sills and horizontal timbers. Pressure against each cross-timber was taken by two diagonal braces resting in notches. The structure was obviously designed for prying out the cross-braces when danger approached. But evidently this practice was abandoned (with the spread of "safety first"), as it must have been inconvenient to escape quickly enough, since now the braces are blown out by two sticks of dynamite in the notches, fired simultaneously from a safe point on shore. The scheme is simple and is reported to work perfectly.

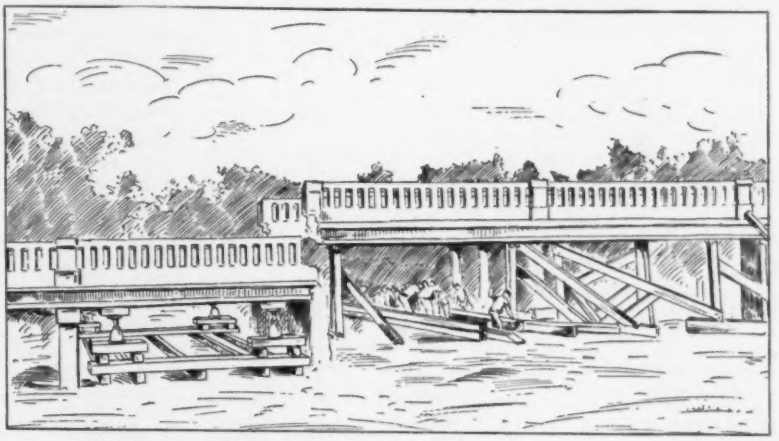


NEEDLE DAM ON WISCONSIN RIVER

⋇

Raising a Concrete Bridge with Screwjacks

In raising seven spans of a concrete-girder bridge over the Bical River in the Province of Ilocos Sur, Luzon, P. I., which had been damaged by high water in 1913, ordinary screwjacks were employed. As the river bed was dry during the season of rebuilding, the jacks were placed on blocking leveled-up under the spans to be raised. One span at a time was raised, the jacks being placed one at each end of each of the five girders of a span. The jacks were turned up in 2-in. lifts. The screws were 2½ in. diameter and 16 in. long. The jacks were reblocked after every 10-in. rise, and great care was taken at every 2-in. interval to make sure that the span moved uniformly. When the span had been pushed to the new elevation, the load was transferred to posts and the jacks were moved to the next span. The piers were then built up to the new elevation and allowed to set 72 hr., when the posts were removed from underneath the spans and the load carried thereafter on the new piers. The work was accomplished successfully, no serious cracks occurring in the concrete. Frank P. James, who designed the original bridge, was in charge of the rebuilding, and he describes it in detail in the January, 1915, *Quarterly Review* of the Philippine Islands Bureau of Public Works.



METHOD OF RAISING THE BICAL BRIDGE, PHILIPPINE ISLANDS

NOTES

Cubic-Yard Table for Concrete Slabs and Walls—The yardage in a slab whose thickness is given in inches and other dimensions in feet must be computed by way of division by 12 and 27. To save the dividing a little table of coefficients has been computed by C. E. Schirmer, Toledo, Ohio. For given slab thickness, multiply the slab area in square feet by the tabular coefficient, and get the contents in cubic yards.

YARDAGE COEFFICIENTS FOR SLABS

(Area of slab in square feet, times coefficient, gives cubic yards contents)

Depth	Coefficient	Depth	Coefficient	Depth	Coefficient
6 in.	0.0185	8¼ in.	0.0255	10½ in.	0.0324
6¼ in.	0.0193	8½ in.	0.0262	10¾ in.	0.0332
6½ in.	0.0200	8¾ in.	0.0270	11 in.	0.0340
6¾ in.	0.0208	9 in.	0.0278	11¼ in.	0.0347
7 in.	0.0216	9¼ in.	0.0285	11½ in.	0.0355
7¼ in.	0.0224	9½ in.	0.0293	11¾ in.	0.0363
7½ in.	0.0231	9¾ in.	0.0300	12 in.	0.0370
7¾ in.	0.0239	10 in.	0.0309		
8 in.	0.0247	10¼ in.	0.0316		

Gasoline Engines Make Butter Smell—Gasoline-engine power in Wisconsin dairies is causing trouble from oleaginous flavors of butter and cream. An investigation by the dairy department of the University of Wisconsin has shown that some buttermakers keep their engines in the vat and churn rooms, or that the engines are too close to the cream separators. It has been recommended that (1) the engine be kept in a room separate from that in which milk, cream and derived products are handled, (2) fuel and lubricants be stored at a distance, (3) drafts from the engine room to creamery should be prevented, (4) the engine exhaust line should end high enough outdoors to prevent any spent gases returning.

A 26-Part Tackle with 7000-Ft. Hoist Lines forms the main fall rigging of the erection traveler used on the Hell Gate arch bridge. A few pieces of 175 tons erection weight and a number weighing over 100 tons have to be handled, those of the east half being already placed. The tackle consists of two 13-sheave blocks. The great length of line is needed to enable the traveler to hoist direct from the river.

The boom-fall rigging has 36 parts of wire rope rigged tandem, and is really two separate tackles side by side and connected to an equalizer. Each top block has six main sheaves and four smaller ones. Each bottom block has five main sheaves and five smaller ones. The two ends of each tackle lead to drums, so that there are four lead-lines, going to two drums each with a partition in the middle.

Curing Concrete Roads by Ponding—Where water is scarce the curing of concrete roads by ponding, or damming puddles of water on top of the road, is recommended by the engineer inspectors of the Universal Portland Cement Co. The green concrete is covered with canvas for the first 10 or 12 hours, by which time it is hard enough to allow earth dams to be constructed. These are built across the road at 25-ft. intervals and also along each side except where there are curbs. Flooding is done in the evening after concreting is finished for the day and water is no longer needed for the mixer. The water is kept at a minimum depth of about 2 in. and allowed to remain on the road for 10 days, after which it is drained off, the earth dams removed and the road opened to traffic.

Do Surveyors Use Traverse Tables?—If one looks through old books on surveying it will be found that traverse tables are almost invariably included. So far as we have been able to find out, all existing published traverse tables are of ancient origin. "Engineering News" frequently has inquiries for good traverse tables, but it would seem that they are no longer so universally used as in the palmy days of land surveying, when it was common practice to "survey" 64,500 acres in two days (Pennsylvania archives). A correspondent wishes to know if there is a present demand for traverse tables computed to 10″, and we will welcome expressions of opinion. The old tables were usually computed to ¼° of angle.

Twelve-Inch Curtain Walls in steel skeleton construction are to be made possible by a revision of the New York building-code section dealing with masonry construction. This section has been revised in the course of the general code revision, and will shortly be laid before the Board of Aldermen for adoption as an ordinance. One of its important improvements over the old code is a clause permitting the use for the full height of the building of 12-in. curtain walls in steel skeleton structures where the walls are supported at each floor by the steelwork. The present law requires a considerable increase of wall thickness in the lower stories, in spite of the fact that the wall does not carry itself. The revision also recognizes terra cotta as a permissible facing equivalent to ashlar. Hollow building blocks of terra cotta may be used in the construction of walls in buildings less than 40 ft. high. A provision for rubble-stone walls is added; these must be 4 in. thicker than brick, stone or concrete walls, and at least 18 in. thick, but in all events limited to 60 ft. in height.

Practical Formula for Switch Lead—Mr. Wentworth's formula in "Engineering News," Apr. 15 (p. 734), prompts me to submit the following practical formulas for switch leads:

Railways use switch points and frogs of standard lengths. The distance then required in simple turnouts would be from heel of switch to toe of frog, which is found by multiplying the frog number by 6.

Then

1. (Frog No.)^2 × twice gage = radius of switch lead.
2. 5729.6 ft. ÷ radius of switch lead = degree of curve.
3. Deg. of curve × $\frac{\text{(switch lead chord)}^2}{(62)^2}$ = middle ordinate in inches for string lining.
4. Three-fourths of middle ordinate = quarter ordinates.

A track foreman can use these formulas.—John J. Malone, N. Y., N. H. & H. R.R., Box 107, Southampton, Mass.

Automobile Operations in Irrigation Service—Five automobiles have been used by Imperial Water Co. No. 1 during the past five years and have been run a combined total of 92,095 mil. during that time. Ray S. Carberry, of Imperial, Calif., Superintendent of the company, gives the following tabular summary of the operations of the cars in his report for 1914:

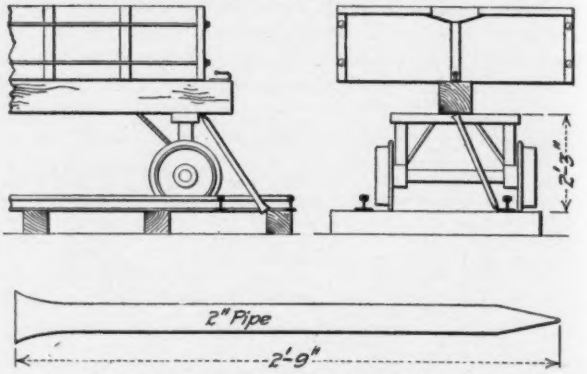
	Overland Cars			Hupmobile	Buick
	1910	1912	1914	1912	1914
Purchase price	$1275.00	$1100.00	$1225.00	$828.70	$1060.00
Miles run	19,447	22,858	13,000	26,730	10,060
Gasoline consumed, gallons	1,966	1,844	1,128	1,631	685
Total repairs	$1123.39	$800.49	$349.95	$626.01	$125.65
Total operating cost	1860.22	1410.85	585.94	1150.98	308.10
Operating cost, per mile	0.0956	0.0617	0.0450	0.0413	0.0306
Total expense	3284.17	2413.08	1485.58	1889.25	1018.58
Total cost per mile	0.1688	0.1054	0.1142	0.0708	0.1012
Average miles per gallon gasoline	9.89	12.39	11.52	16.39	14.69
Average cost per mile, casings	0.0225	0.0187	0.0126	0.0116	0.0137
Depreciation on car, per mile	0.0511	0.0250	0.0558	0.0175	0.0556

The Overland, 1914 model, and Buick, 1914 model, are still in use, and valued at $500 each.

Water-Supply Hand-Pumps in the Philippines—A large number of public artesian wells have been put down by municipalities in the Philippine Islands with the financial aid of the Insular Government. Many such wells are nonflowing and the municipality is usually unable to do more than install a hand-pump. The public use has been so severe that the various officials responsible have generally been unable to keep the pumps running over one-half the time. A solution of the difficulties has now been worked out by the Insular Bureau of Public Works and is noted by J. W. Vickers in the January issue of the department's "Quarterly Bulletin." This involves (1) standardization of pump equipment, (2) training local workmen to make minor repairs, and (3) placing responsibility for repair work on a single person. In the standardization work it is recommended that there be only two pump types—(a) a deep-well submerged design for cases where the level is 20 ft. or more below ground, and (b) a suction design for lifts of less than 20 ft. Practically all the well-known and satisfactory American pumps are now seen in this service, but their maintenance has been found difficult under the severe service, lack of spare parts and unskilled attention.

Simple Device for Rerailing Dumpcars—The quick removal of spoil is often vital to rapid excavation. Every time a dump-car is derailed it means the tying-up of the spoil-disposal system. To reduce such delays to a minimum at the Kennerdell (Penn.) tunnel now nearing completion on the Allegheny Valley division of the Pennsylvania R.R., a simple and successful contrivance was employed.

The device consisted merely of a piece of 2-in. pipe 2 ft. 9 in. long with ends flattened at right angles. To rerail a car one end of the pipe was braced against the cross-tie while the other end was pushed against the underframe of the car. Force was applied to the car against the pipe; the car would



PIPE RERAILER IN SERVICE

lift in the air with one end balanced by the rerailer until the car lost its equilibrium and dropped onto the rails. The pipe was of the correct length to cause this.

This device is so simple that it probably has been used elsewhere. However that may be, it is thought very highly of by the builders of the Kennerdell tunnel. The John A. Kelley Co., of Philadelphia, is the contractor. George Patton is Resident Engineer for the railroad.

Stringing Heavy Overhead Electric Conductors—Street cars are being used by the Bush Terminal Co., of Brooklyn, N. Y., for pulling into place several pairs of 1,000,000-circ.mils conductors. The distance is several blocks. According to the April "Brooklyn Bulletin," published by the Edison Electric Illuminating Co. of Brooklyn, the poles along this line are cast iron with metal crossarms, the spans being about 90 ft. each. Before attempting to install the cables the crossarms are wrapped with burlap to prevent abrasion. The cables are then attached to a long lead line carried over the arms of several poles and attached to a street car. It should be stated in this connection that the Bush Terminal Co. operates a private street railway.

Editorials

In connection with the numerous signs of reviving activity in business it is of much interest to note that the Construction News Supplement of our last week's issue contained the largest number of items ever published in that department. The total number of items was 832. Of course, this is not an accurate method of measuring engineering activity, since one item may relate to work involving a million dollars' expenditure and another to work involving only $50,000. The items themselves show, however, that there is work going on in cities and towns scattered all over the country, the aggregate of which is of large proportions. Certainly, the activity in engineering work is much greater and the outlook for the future is far brighter than seemed at all likely six months ago.

By Guess or by Meter

Although the last decade has witnessed much progress in the introduction of water meters, by far the larger part of the water supplied in the United States is still sold by guess. This is absurd and unjust, and it encourages that useless and almost criminal waste of water for which American cities are notorious. Selling water by guess encourages waste of life as well as of property, because it renders increasingly difficult the problem of providing sufficient quantities of pure and wholesome water, under adequate pressure. Water-works officials are a unit in favor of selling water by meter instead of by guess. The growing demand for efficiency and economy in municipal affairs will aid the movement already well under way to sell water by meter measurement only.

Filing Contract Plans and Specifications in "Engineering News" Office

We deem it proper to call attention in this place to the advantages to both engineers and contractors of the facilities afforded in the office of ENGINEERING NEWS for the filing of plans and specifications on work for which bids are advertised and on which general competition is desired. A very heavy expense is incurred by contractors in traveling from place to place where work is to be let, to examine the plans and specifications, which are usually filed in the office of the engineer for the city or company undertaking the work.

New York City is by far the greatest headquarters for contractors in this country. Plans and specifications which are open to the inspection of bidders in New York City are made available to many contractors who would not incur the expense of a journey to the site where the work is to be performed merely on the chance of securing the contract.

It may be said that contractors generally desire to inspect the site where work is to be carried out before bidding on it. If the plans and specifications are on file in New York, however, the contractor having an office or a representative there can examine them and determine whether it is worth his while to visit the site and submit a bid.

There is a very large class of contracts, moreover, on which the plans and specifications tell the whole story. We refer to contracts for machinery and supplies. New York is by far the largest market in the country for everything of this sort, and purchasers filing their plans and specifications for such contracts in our office can secure the competition of bidders who would not make a journey to some other city.

This service, which ENGINEERING NEWS renders to its readers free of charge, is of especially great value in connection with foreign contracts. Engineers in any part of the world, desiring to make important purchases of engineering materials or machinery on plans and specifications, can forward a set to this office for filing and thus make them available to a large circle of responsible bidders.

Damages for Water-Borne Typhoid Fever

Private water companies in New Jersey and elsewhere will do well to take careful note of the decision rendered in April of this year by the State Supreme Court of New Jersey. The case was on appeal from a verdict of $750 awarded B. Henry Jones against the Mount Holly Water Co. to recover expenses and loss of time due to the illness of three of the plaintiff's children from typhoid fever or paratyphoid, beginning in January, 1912, during an outbreak of typhoid in Mount Holly. The plaintiff had paid the company $29.50 in advance for a year's water-supply, covering the period during which the illness occurred.

The court, in reviewing the testimony and law of the subject, held that there was evidence that the water-supply was polluted with sewage and that the company had known of the evidence for three and a half years prior to the epidemic. It also held that there was a contractual duty on the part of the company to supply the plaintiff with pure and wholesome water. Two paragraphs expressing this view may be quoted from the decision, as follows:

> **It must be borne in mind that the defendant company was in the water-supply business for profit. The plaintiff had paid for the supply which he was to receive in advance. Hence, it became the duty of the defendant company to give to the plaintiff water fit for domestic purposes, including fitness for drinking.**
>
> **Water is a necessity of life and one who undertakes to trade in it and supply customers stands in no different position to those with whom he deals than does a dealer in foodstuffs. He is bound to use reasonable care that whatever is supplied for food or drink shall be ordinarily and reasonably pure and wholesome.**

The court cites approvingly a clause in the contract between Jersey City and Flynn (the Jersey City Water Supply Co.) which provided that the water delivered to Jersey City "should be pure and wholesome and free

from pollution deleterious for drinking and domestic purposes." The court also cites with approval a declaration (Peffer V. Penn. Water Co., 221 Penn. 578; 70 Atl. Rep., 870) "that pure and wholesome water necessarily means such as is reasonably free from bacteria and coli, or any other infection or contamination which renders water unfit for domestic use and unsafe and dangerous to individuals." Moreover, it was necessary for the plaintiff to do no more than make it "reasonably appear that the drinking water was the probable efficient cause of the typhoid fever."

Compared with many of the earlier decisions in cases of like general import this one certainly shows that much progress has been made toward holding a water company financially liable for sickness caused by the distribution of unsafe water for domestic uses.

The Late George S. Morison and the Panama Canal

In ENGINEERING NEWS, Mar. 11, 1915, p. 497, in commenting on the recognition by Congress of the service rendered by the engineer officers on the Panama Canal work, we referred to the service rendered by the late Alfred Noble and a number of other civilian engineers in connection with the original selection of the Panama route for the canal. We made no attempt to make the list complete, for there were many engineers who deserved credit in connection with that important decision.

Our attention has been called, however, to the prominent part played by the late George S. Morison in the selection of the Panama route. Those who know something of the inside history of the Isthmian Canal Commission of 1899-1901 state that Mr. Morison was probably the most influential man on that commission in securing the adoption of the Panama route.

As many of our older readers will recall, that commission was made up of the members of the previous Nicaragua Canal Commission, Admiral Walker, Colonel Hains, and Professor Haupt; and of Alfred Noble, who had served on the Nicaragua Canal Commission of 1895; Colonel Ernst, of the Corps of Engineers; Messrs. George S. Morison and William H. Burr; Senator Pasco of Florida, and Prof. Emory R. Johnson.

Of all these men Mr. Morison had the most dominating and forceful personality. His professional work, it is true, had been largely confined to the field of bridge engineering, but he owed his engineering success, not to his great technical knowledge, but to his commanding personal qualities. He was personally the very antithesis of Mr. Noble, who was quiet and retiring in disposition and likely to be unappreciated save by those capable of recognizing his remarkably sound judgment.

In the reorganization of the commission Mr. Morison undertook the investigation of the Panama route. Under these circumstances he would be extremely likely to be the leader in the advocacy of that route in the commission. There is little doubt that some of the other members of the commission were strongly disposed, at the outset at least, to favor the Nicaragua route. The task of convincing them of the superior advantages of the Panama route naturally fell to Mr. Morison, and that he was fully competent to perform this task no one who knew him would doubt.

Mr. Morison was one of the great engineers of his generation, and it is proper that his important contribution to the Isthmian canal enterprise should be known.

Longer Cast-Iron Pipe

Among the exhibits at the convention of the American Water Works Association a year ago perhaps none attracted more attention than the 16-ft. lengths of cast-iron pipe shown by the American Cast-Iron Pipe Co. At that time only a few cities had tried the pipe, but now it has been laid or ordered by scores, if not hundreds, of cities and companies.

Inquiries which we have made have brought out expressions of almost universal satisfaction with the 16-ft. length of pipe, and no out-and-out disapproval. Truth compels us to say, however, that most of the users who responded to our inquiries do not seem to have kept such records as would enable them to make an exact comparison of the merits of the 16-ft. and the 12-ft. lengths.

The advantages of the 16- over the 12-ft. lengths of pipe, as stated by a number of water-works superintendents and contractors, are: 25% less of (1) lengths of pipe to unload from cars, load to and unload from wagons and lower into the trench; (2) bell holes to dig; (3) joints to calk; (4) lead and jute to supply. To these savings there is added by W. S. Rathell, Supervising Engineer, Texas Power & Light Co., Waco, Tex., 25% less leakage hazard.

On a job consisting of 18,000 ft. of 4-in. and 3000 ft. of 6-in. pipe at Douglass, Kan., Joseph O'Neil, of the O'Neil Construction Co., Leavenworth, Kan., stated that the dry hard digging encountered made the labor saving due to 25% fewer bell holes very welcome.

J. R. Sutherlin, of the Sutherlin Construction Co., of Kansas City, Mo., states that his company has used only 16-ft. lengths since these were available and that the saving in laying is about 2c. per ft. This agrees fairly well with the experience of C. O'Leary, Superintendent of the Light, Water and Gas Department of Newton, Iowa, who laid 41 lengths (492 ft.) of 4-in. cast-iron pipe, each 12 ft. long, for $65.87, or 13.4c. per ft., and the same number of 16-ft. lengths of 4-in. pipe (656 ft.) for $76.01, or 11.6c. per lin.ft., including all labor and all jointing material, but not the pipe itself.

Two reports agree in stating that the saving in labor, lead, etc., for the 16-ft. pipe is just about offset by the extra price charged for the pipe. Thus W. O. Bickelhaupt, of the Des Moines Iron & Bridge Co., says that on machine-trench work the cost of laying 16-ft. pipe "has been just a trifle" less than that of laying 12-ft., but the pipe costs more and the "breakage is greater." Cost figures for hand-trench work are not yet available to this company. Likewise the N. S. Sherman Machine & Iron Works, of Oklahoma City, Okla., says that the "premium of $1 a ton on the 16-ft. pipe," the "additional cost of handling, and the increased amount of breakage practically offset the saving in the lead joints and bell holes." This company also states that the 16-ft. pipe requires more care with the trench grade than does the 12-ft.

So far as the replies state, most of the 16-ft. pipe has been used on straight or relatively straight runs. One man said he had had no trouble with it on curves and another thought that shorter lengths would have to be substituted on curves.

While, as we have already stated, few complete data are available, the general consensus of opinion is favorable to the 16-ft. lengths of pipe. For the smaller diameters and under favorable conditions it seems likely that the use of the longer pipe lengths will prove to be advantageous. Reliable cost data on the subject would be welcome.

Some Notable Tests of Car Resistance

There are plenty of data available in railway literature on the resistance of railway trains. Few experiments have been made, however, to determine the resistance of individual cars. The practical value of such tests will be evident to every engineer who has the responsibility of constructing a freight yard with one or more tracks down which cars are to run by gravity. The grade must be steep enough so that a car will not come to a stop upon it, but not so steep that the car will reach a dangerous speed in case it is running free without a brakeman. The problem comes in on a large scale in the design of gravity sorting yards.

To obtain accurate data for use in constructing such yards an elaborate series of tests has been carried out by Prof. C. L. Eddy, of the Case School at Cleveland. The tests were made in the gravity sorting yard of the Lake Shore & Michigan Southern Ry. at Collinwood, Ohio. A full description of the tests is given in a paper by Professor Eddy in the *Bulletin* of the American Railway Engineering Association for March.

The method of making the tests was simple. Cars which were pushed over the hump in the ordinary routine operation of the yard had their velocity accurately determined to 0.01 sec. by an electric chronometer as they passed two different points on the south ladder track about 400 ft. apart. By comparing the change of velocity of the cars in passing over this stretch of track with the force imparted to the car by the downgrade of the ladder track, 1.135%, the resistance of the car was accurately determined. The speeds of the cars were generally low, the range being from 8 to 15 mi. per hr. The wind velocities at the time of the test were generally not over 20 mi. per hr.; hence the air resistance to the motion of the car was in general small and could practically be omitted from consideration.

Passing now to the discussion of the results obtained from the tests, the first notable feature is the high average resistance observed. This average was 21.7 lb. per ton in one series of tests and 22.3 lb. per ton in the other series, made a year later. A part of this high resistance is explained by the fact that the cars were running down the ladder track over switches and frogs, where the resistance is undoubtedly more than on ordinary unbroken rail. Another factor, probably, in producing the high resistance is that the cars had run only a short distance at the time their speed was taken. It may be assumed that the cars to be sorted had been standing in the receiving yard long enough at least to have the journals cool off before they were pushed over the hump by the drilling engine. Various tests indicate that car resistance is materially reduced after a car has run far enough to have its journals well warmed. A set of observations made when the thermometer was at 20° F., which were not included in the average given in the preceding, showed a car resistance of 26.8 lb. per ton, conforming to the well-known rule that car resistance increases as the temperature lowers.

All these figures for resistance, as stated before, are exceedingly high compared with the formulas in general use for computing train resistance. For example, Henderson's formula for the resistance of trains behind the tender at a speed of 10 mi. per hr. is $R = 3.5$ times the weight of the train in tons + 50 times the number of cars in the train. The second term in this formula is added to allow for the increased resistance of empty cars over loaded cars. By this formula the resistance of a single car weighing with its load 50 tons would be 4½ lb. per ton, or less than one-fourth that shown by Professor Eddy's tests. Other formulas in general use give a train resistance of only 5 to 7 lb. per ton for modern high-capacity cars. There are, in fact, no train-resistance tests on record which show any such high resistance to the movement of the car as is indicated by these tests of single cars by Professor Eddy.

Besides the high resistance of the cars tested, the wide variations in the resistance of the cars is a noteworthy feature. A few of the cars showed very low resistance, one as low as 4½ lb. per ton and another 6 lb. per ton. These results were so far below all the other results obtained that some error may possibly have occurred in the observations or computations. A large number of the cars tested, however, showed resistances of from 11 to 16 lb. per ton.

At the opposite extreme, certain cars showed phenomenally high resistance. One car showed a resistance of 41.8 lb. per ton, another of 36.3 lb., another of 35.6 lb., while several were above 30 lb. It is comparatively easy to account for these cases of exceptionally high resistance. The brake rigging may have been stiff enough to keep the shoes pressed against the wheels, or trucks out of square may have caused the flanges to hug the rail and produce particularly high resistance in running over the switches and frogs.

That a considerable variation in resistance of cars does occur is a matter of common observation in the operation of gravity switching yards. Probably the largest element in variation of car resistance is journal friction. It is known from both laboratory tests and from practice that a car journal in first-class condition with a well-fitting bearing, thoroughly lubricated, will run with an extremely low coefficient of friction when once the film of oil is established between the journal and the bearing. On the other hand, it is well known that under the actual conditions of railway-freight-car operation, with the lubrication and packing of journal boxes performed at irregular intervals, with wide variations in the condition of the journal, the wear of the bearing and its adjustment, there is a wide variation in the friction of car journals. It is probable that these variations alone would be sufficient to account for most of the range between the higher and lower values found in Professor Eddy's tests.

In analyzing the results of his tests Professor Eddy concludes that in gravity classification yards, if it is desired to insure that there shall be no slackening of speed of the cars while running down the ladder from which the classification tracks lead, the grade of the ladder should be 1.65%. If this were done, however, it would be necessary to apply brakes on nearly all the cars to control their speed and prevent them from striking the turn-out

curve at a speed too high for safe operation. On the Collinwood ladder track, with its grade of 1.175%, cars with the average resistance ran down the ladder at practically uniform speed. Cars of higher resistance than the average slackened speed and the cars of lower resistance gained in speed. There were, however, very few cars tested whose speed increased to materially over 15 mi. per hr. in running down the ladder. On the other hand, the cars which slowed down did not slacken speed enough to offer any serious hindrance to operation; and it may be questioned, therefore, whether it would be wise to increase the ladder track grade to the extent suggested by Professor Eddy.

Bathing in Water-Works Reservoirs

The pleasant practice—for the bathers—of taking a dip in reservoirs from which city drinking water is supplied has given much concern to water-works authorities in various parts of the country. Of all the states of the Union where one would least expect trouble from that source Massachusetts comes first. Yet a complaint of wholesale bathing in a reservoir is recorded in the last annual report of the Water Board of Taunton, Mass.—and, surprising to relate, state officials were responsible.

The facts were these: Officers of the state militia selected an old camping-ground on or near one of the Taunton reservoirs as a site for a temporary camp. The officers asked the water-works authorities if they had objections, and were promptly given an affirmative reply, through the Governor, with a request for a hearing. The decision to use the site was made before a hearing was granted. Assurances were given the Water Board that the water-supply would be protected and that safety regulations had been made by the Engineer of the State Board of Health. The rule which he laid down was that guards should be "placed to prevent the danger of soldiers bathing in the pond."

The camp was opened, but no attempt was made by the military authorities to protect the water-supply. On the first warm day, 300 "soldiers" bathed in one of the ponds. The next day, after 125 men had gone into the pond, guards were set. No further bathing took place at the camp site, but it was reported that there was bathing elsewhere in the reservoir at night.

From the facts cited the Water Board draws the conclusion "that the city cannot rely upon the law or upon the coöperation of others to protect its reservoirs from pollution so long as the public has access to their shores. Ownership and fencing off the shore front are considered essential to safety."

This conclusion accords with general experience and a rapidly growing practice. In fact, many cities are not stopping short with the acquisition of reservoir shores, but are acquiring large portions and sometimes virtually the whole of their water-works drainage areas. An interesting example of large acquisition of this kind is afforded by Newark, N. J., as described elsewhere in this issue.

Letters to the Editor

No Water-Works for Him

Sir—The inclosed clipping from a newspaper in this state is typical of the attitude taken by the more ignorant people on public improvements and shows the general tendency to believe that all engineers, contractors and purchasers of bonds are grafters. It is an amusing letter and if you care to use it in your paper you may do so.

W. L. BENHAM.

Oklahoma City, Okla., Apr. 3, 1915.

The letter, spelling and all, is as follows:

Editer uf the Demercrat:

Plese alow me to space in yur valable paper to say a few words about the water situashion here in Wilburton. First thing that that will did after voting the bonds will be to give ingerneres a slice of the $100,000; then the bond men will git theirs; then the land oners will git theirs, more than the land is worth; and we we will have about $85,000 left to build the plant. And we can't tell just yet how many other grafters will git a slice. But we'll bet a coon skin they git in al right in alright. I would be in favor of gitting rid to the grafters before we vote any king of bonds. Seconb: How manf many of us would cut loose from the present pipes and hook onto the new when there would be about $10 to $15 to boot? As long as we have as good watter as we have now, I am going to keep my pipes hooked up just like they are.

Third: If we vote these bonds they tell me that our tax would be outten sight. It looks to me just like a plaster being put on a feller's house and lot for 25 years.

I am not a writter and don't clame to know much, but I do hope the people will not make us pay this much for watter.

Hopin you will be abel to read this and will print such as you see fit, I beg to remain, yours,

truly,
A Tax Payin Citizen.

[Although sentiments like those expressed are far too common, happily their holders are not, or do not long remain, in the majority, else nearly every "city" with a population of a thousand or two in most parts of the country would not be provided with water-works.—EDITOR.]

The Philadelphia Water-Works Handicap: 1804-1914

Sir—In your issue of Mar. 11, 1915, discussing "Philadelphia Water-Works Problems," you say:

For two-score years or more the successive administrative heads of the Philadelphia water-works have operated under a heavy handicap. . . . Various parts of the works, upon which important sections of the city are dependent for their supply, are liable to be put out of commission at any time because funds are not provided by the appropriating authorities . . . financial and other reforms are needed before the Philadelphia water-works can be put on a thoroughly safe and economical basis.

I submit two quotations, the first of which throws light upon the nature of the handicap mentioned and the number of scores of years during which it has been carried, while the second seems to point out the "financial and other reforms needed."

So far back as 1804 (5½-score years ago), when the Philadelphia city councils were supposed to be made up of representative and worthy citizens, Benjamin H. Latrobe, the designer and builder of our first water-works, speaking from bitter experience, wrote to Nicholas I. Roosevelt (the builder of the engines, brother to Col. Roosevelt's grandfather, and later Latrobe's son-in-law):

> First the sub-committee of the Watering Committee must assent to an arrangement, then comes the Watering Committee itself, then the Common Council, and the Select Council—all avaricious, unjust, ignorant and proud.

Mischievous intermeddling by an incompetent and cumbersome legislative body with the plans and operations of supposedly expert officials has been the bane of our entire public-works system from the beginning. Evidently the framers of our city charter recognized this fact, and sought to remedy it by placing the design and execution of public works in the hands of men presumably qualified, and by restricting the legislative body to its proper function of finding the means necessary for carrying out the administration's plans; for that document provides (and this is my second quotation):

> Water-works, . . . structures of every kind for public use . . . shall be under the direction, control and administration of the Department of Public Works. . . . Councils shall, by general ordinances, provide for the proper and efficient conduct of the affairs of the city by the Mayor and several departments, . . . but they shall not pass any ordinances directing or interfering with the exercise of the executive functions of the Mayor, Departments, boards or heads, or officers thereof.

Yet, whenever so ordered by "the real masters, who stand erect behind the throne," their honorable bodies of butchers, bakers, candle-stickmakers and so-called politicians refuse to "provide for the proper and efficient conduct of the affairs of the city by the Mayor and several departments," and they do "pass ordinances directing and interfering with the exercise of the executive functions of" the administration elected by the people for the purpose.

JOHN C. TRAUTWINE, JR.

257 S. Fourth St., Philadelphia, Apr. 10, 1915.

Stresses in Mill-Building Bents

Sir—In your issue of Apr. 15, 1915, p. 740, W. L. Six states, with reference to the reactions to wind-pressure acting on mill-building bents, that the suction on the leeward side may amount to about 45% of the total wind action, which would tend to show that the assumption of equal reactions is nearly correct.

The writer remembers having seen elsewhere the suction on the leeward side rated as high as this, but believes that this refers to sharp-edged plates of comparatively small dimensions. Assuming that a suction of the same intensity as the direct wind-pressure occurs on the leeward side as far as 2-3 ft. from the edge and that the diameter exposed to the wind-pressure is 8-10 ft., the above estimated value may be correct. The suction will, however, have very little influence on a mill building 50 ft. high, 60 ft. wide and several hundred feet long. Assuming, for the sake of the argument, that the suction on the leeward side of the mill building is 50% of the total wind force, then the reactions are equal and are $\frac{1}{2}\ P$, but the bending moments at the kneebraces are also equal and are $\frac{1}{4}\ Ph$, which is even smaller than the $\frac{1}{8}\ Ph$ that I suggested. The suction must be understood as a uniformly distributed force acting on the leeward side of the building.

Returning to the main question: The aggregate of the bending moments on both columns at the kneebraces is $\frac{1}{2}\ Ph$. To assume, as is ordinarily done, that this moment occurs at the kneebrace of the leeward column is to assume that there is no moment at all at the kneebrace of the windward column—which is inconsistent with the elastic theory and instantly strikes people as unreasonable, when their attention is called to it.

C. L. CHRISTENSEN.

2122 Alpine Place, Cincinnati, Ohio, Apr. 19, 1915.

Are These Contractors Breaking the Sherman Law?

Sir—I was recently told that in a certain Eastern city the general contractors have formed an association, and when a large job comes up, no more than ten are allowed to bid on it. Each man adds $500 to his bid, and the successful bidder, as soon as he is awarded the contract, pays each of the nine others $50 each, which compensates them for the time employed in figuring on the job.

My informant advised me that it was surprising how close the total bids ran under this system. It will readily be seen that even if a contractor did not get a single job during the month, but had an opportunity to figure on four, the $200 received would at least pay his overhead expenses for that period.

An organization of the kind described looks to me thoroughly feasible, for when a contractor was found to be abusing the privileges, he could be summarily dismissed from the organization with a certain amount of publicity and, in the event of an architect or engineer allowing him to figure on any of his work, the rest of the contractors in the organization could then refuse to figure on the job or any other job where similar conditions existed.

CONTRACTOR.

Middle West, Apr. 25, 1915.

[Possibly, the contractors who are associated in this combination think that if they do not combine on the bidding price they will not be within the terms of the Sherman law. That seems, however, like skating on very thin ice. There have been several cases recently, indicating that the Federal authorities are closely watching the activities of trade associations. Unless the members of the association choose to run the risk of indictment and imprisonment, they would do well to seek an "immunity bath" from the new Federal Trade Commission or, failing that, to abandon their scheme.—EDITOR.]

Asphalt-Block Pavements in the Borough of Manhattan

Sir—In ENGINEERING NEWS, Apr. 1, is a letter disagreeing with some of my statements in the issue of Mar. 18, under the title, "New York City Experience with Asphalt Block Pavements." For the title [which was written by the editor] apologies are due for implying boroughs other than Manhattan, although the error is a very common one and still persisted in by the U. S. Post Office Department.

It should be brought to the notice of such of your readers as may be interested that the article was an abstract of part of the introductory remarks in my Annual Report for the year 1914 of the Bureau of Highways of Manhattan. For detailed records and tabulations from which my conclusions were drawn readers are referred to my report in full and to those for 1912 and 1913. (The 1914 report is not yet published, but the two others may be obtained by application to the office of the President of the Borough of Manhattan.)

HENRY WELLES DURHAM.

New York City, Apr. 16, 1915.

The U. S. Geological Survey and the Topographical Survey of Cincinnati

Sir—I was keenly interested in your editorial on "Competition of a Government Bureau with Private Engineering Practice" and the correspondence on the same subject in your issue of Apr. 22.

As I was in charge of the sewerage investigation in Cincinnati, which included the topographic survey referred to in the last paragraph of the letter from the Director of the U. S. Geological Survey, it seems entirely proper that I should supply the facts in my possession which bear on this portion of the discussion.

The Topographic Survey of Cincinnati covered 102 square miles. The firm of Brown & Clarkson performed all of the planetable work. Their contract at first included but 70 square miles, the area of the city. Later, it was decided to map 32 square miles outside the limits of the city. At that time it seemed impossible for Brown & Clarkson to take on this additional work and complete it within a specified time. So, with the full knowledge of Brown & Clarkson I took up with the officials of the U. S. Geological Survey the question of obtaining assistance from that bureau. We asked for a loan of instruments and men, with the full understanding that we would pay the salaries and expenses of these men. In no sense was our proposition a request that the U. S. Geological Survey compete with private engineers.

However, assistance of any sort was refused, and further than this, it was clearly stated in two letters that it was the policy of the U. S. Geological Survey not to enter into competition with private parties. Inclosed are copies of these letters, for your information. There is nothing in them qualifying the refusal of coöperation to the City of Cincinnati on the basis that the area to be mapped was small or that it had been previously mapped, but on the other hand, it was distinctly stated and reiterated that it was against the policy of the Survey to enter into competition with private parties.

As a matter of fact, after the Cincinnati work was started the Geological Survey had two topographers revising the Cincinnati sheet. These data might well have been secured through the Cincinnati survey. Also, in spite of the refusal of the Survey to coöperate with us even to the extent of reproducing our survey in return for the use of data, reproductions of the survey, which included 48 engraved sheets, have been given to the U. S. Geological Survey and are being used in connection with the preparation of a geological report on the Cincinnati district.

Personally, I indorse most heartily this policy of no competition with private engineers and am well satisfied that at the time our request was made the Survey was in no position to give us the assistance desired. We finished our work in good shape and on time and have no complaint to make, but the apparent abrupt reversal of policy in the Texas case is very difficult to understand, and it seems unfortunate that the Survey has not consistently adhered to its policy of "refusing to utilize its organization to compete with private engineers."

H. S. MORSE,

Director Cincinnati Bureau of Municipal Research.

804 Neave Building, Cincinnati, Apr. 26, 1915.

Competition of the U. S. Geological Survey with Private Engineering Practice

Sir—There are several points in the letter of the Director of the U. S. Geological Survey written in reply to my protest (published in your issue of Apr. 22, p. 789) against unfair competition of the U. S. Government with private engineers, on which I am sure your readers will appreciate a comment.

While correspondence between the Houston Chamber of Commerce and the Director of the U. S. Geological Survey through the medium of a member of Congress may have been carried on some years ago, the facts remain that a member of Congress is not a county official, nor is the Houston Chamber of Commerce an organization possessing any civil authority; neither has the power to transact county business. Furthermore, while the correspondence referred to was coming to a head, the county authorities themselves, consisting of four commissioners headed by the county judge, were advertising for bids for the making of the needed survey by private engineers. Would these county authorities have gone ahead with their plan for advertising and receiving bids on this work had they been in even *indirect* communication with the Federal authorities? To even imply that such is the case would be to challenge the good faith of the call for bids—a challenge which the writer of this letter considers absurd and preposterous.

As a half-truth is always dangerous, I must amend and amplify that paragraph of the Director's letter in which are mentioned the bids received by the county. The bids received and opened by the county judge on the morning of Feb. 10 ranged from $27,000 to $175,000. The bid submitted by the writer of this letter in coöperation with the firm of Brown & Clarkson was a little over $119,000. This bid was to include the following: A topographic survey of the entire county with 1-ft. contours *located,* except in cut-banks of streams, where the 5-ft. contours were to be located. It included special large-scale surveys along streams; surveys of certain watersheds extending beyond the county lines; collection of drainage data, and the making of a drainage report and plans.

The supplementary offer made by the same organization and referred to in the Director's letter as being some $50,000 under its formal bid was for a topographic survey *only,* with 5-ft. contours *located* and intermediate contours *sketched* in. It did not extend beyond the county lines and did not include drainage report and plans. This offer was, in fact, to make a survey identical with the one proposed by the Geological Survey and at a cost of $15,-

000 less than their estimate of the cost of the work. I believe that this statement clears up Director Smith's apparent misunderstanding of the matter.

The estimate of $80,000 given in the writer's letter of Apr. 8, and also mentioned in the Director's reply, was taken from a newspaper report of a short address given by the Chief Geographer of the U. S. Geological Survey before the Progressive League in Houston. He is credited with having said:

> . . . the work of a topographical survey of Harris County will cost on an average of between $50 and $60 a square mile, or about $80,000 for the entire cost.

If we accept the area of Harris County given by the U. S. Census as being correct, namely, 1,058,560 acres, the cost per square mile given above would make the total cost for the survey of the county between $82,700 and $99,240.

The writer is not familiar with the details of coöperation under which the survey of the Fifth Louisiana Levee District mentioned in the Director's letter was made. He is aware that in an apparently similar case, that of the Tallahatchie Drainage District in Mississippi, the Governor of the state joined the president of the drainage district in entering into contract with the U. S. Geological Survey. If a similar condition obtained with regard to the Louisiana work, or if by act of legislature the Fifth Louisiana Levee District was made a state undertaking, then the work done there comes within the well-established precedents for state coöperation and is not a precedent upon which to hang justification for the Harris County work.

If, however, such is not the case, and the Louisiana work was done under direct contract between the Director of the Geological Survey and the president of the Fifth Louisiana Levee District, then it is not only another case of flagrant competition with private engineers, but, it seems to me, a direct violation of that clause of the organic act creating the U. S. Geological Survey (Mar. 3, 1879), which says:

> And that the Director and members of the Geological Survey shall have no personal or private interests in the lands or mineral wealth under survey, and shall execute no surveys or examinations for private parties or corporations.

Concerning the last paragraph of the Director's letter I can only say that the reason given therein for refusing to coöperate in a topographic survey of the City of Cincinnati is new to the writer and in no way resembles the one given something over two years ago when the writer was Engineer in Charge of the Topographic Survey of 1912 of the City of Cincinnati.

I must close this letter with a question, the answer to which will surely interest all engineers having to do with topographic surveys. In contributing $5000 toward the cost of the Harris County survey as representing the Federal Government's interest in that survey, is it to be understood that this is what it would cost the Geological Survey to make its standard type of topographic survey over that area? Putting it a little differently, are the topographic surveys made by the Geological Survey in other parts of the country where conditions are similar to those in Harris County costing only about $3 per square mile, or is Harris County more a benefactor than a beneficiary?

HUGH C. MITCHELL.

1815 U St., N. W., Washington, D. C., Apr. 23, 1915.

The Status of the Engineer

Sir—It is hard to tell from "Civil Engineer's" letter (ENGINEERING NEWS, Apr. 15, 1915, p. 742) whether his comparison of the offices of his confrères is irony or not.

In making comparisons with the other learned professions one important point is generally overlooked—the limited scope for the independent practicing engineer. Even small villages boast of one or more doctors, and every village of a few hundred inhabitants can support a lawyer; because every person at some time needs the services of a physician and many are obliged to employ a lawyer in times of trouble.

There is little demand for the engineer, however, outside the various public-service corporations and the municipal and state governments, so that the public have little or no personal interest in his work and are too easily led away by the delusive rhetoric of the professional politician. If the worth and work of the engineer were appreciated by the public an enormous amount of annual waste could be eliminated, but until the engineering profession has educated the public into the need for its services we will continue to hear complaints regarding the "Status of the Engineer."

C. D. NORTON.

Sudbury, Ont., Apr. 20, 1915.

Professional Practice by Engineering Teachers

Sir—I graduated from an Eastern university whose professors were sought to do outside work. I never knew of their competing for it. I would not send a son of mine to any engineering college whose professors were not well known in the engineering profession for their engineering work as well as their teaching. They are the only teachers who make theory and practice fit together smoothly.

In order to keep any man at his best there must be adequate compensation for his labor. The college professor who does outside work should be allowed to receive the compensation for it. If a professor gives the required number of hours per week to his teaching, which at most is about forty, what he does with the rest of his time should concern no one but himself. Any regulation upon the extra earnings of a professor is just as contemptible as was the act of the Dean who offered to furnish free student labor. In all but one case that has come under my observation, students doing work on professors' outside practice have been paid; and both the experience and the money were appreciated. Some of my professors I have known to work fourteen hours a day for long periods, and none of them, so far as I know, gave scant time to their students.

J. H. M.

San Francisco, Apr. 13, 1915.

NOTES AND QUERIES

A formula for computing the stresses in large washers—annular disks supported near the outer edge and loaded at the inner edge or over the entire surface—is desired by F. W. Salmon, 516 North 20th St., Birmingham, Ala.

Design Suggested for Shore-Protection Works on the New Jersey Coast

Acting under provisions of the act creating it, the New Jersey Harbor Commission has recently looked into the question of shore protection on beaches of the state, in response to requests for aid from the municipalities of Longport and Stoneharbor.

To this end the Chief Engineer of the Commission, B. F. Cresson, Jr., Jersey City, has submitted a design for shore-protection works to be used under the conditions common to the beaches of New Jersey; that is, a flat sloping beach with comparatively low tidal range, but occasional very high water and exceedingly severe wave action under the impulse of easterly winds. The Chief Engineer submits as his opinion that a comprehensive study of the coast front of New Jersey would be a very profitable thing for the state to undertake, inasmuch as by some well-thought-out system of coast protection large amounts of money might be saved, which are now spent to repair the regular damages done to the structures along the ocean front. The present design, however, is made for immediate use and is the result of a rather complete investigation of domestic and foreign designs for similar conditions.

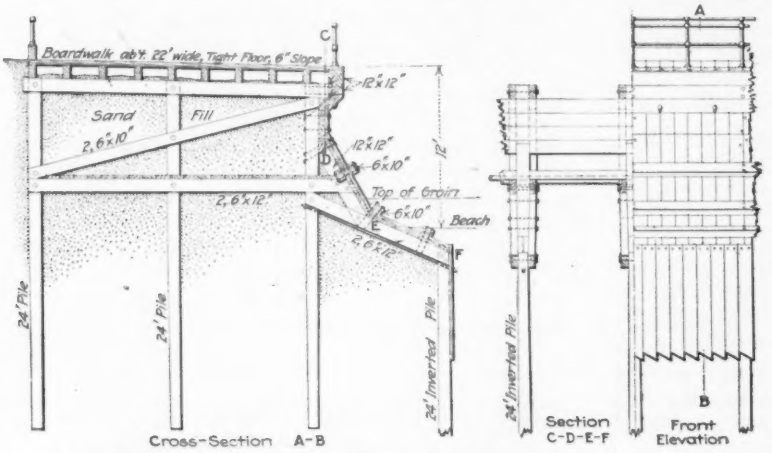


FIG. 1. PROPOSED SEA WALL CONSTRUCTION

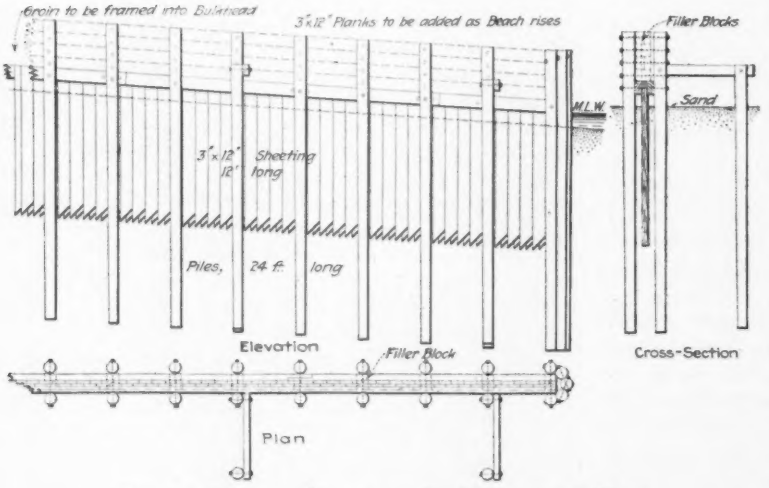


FIG. 2. PROPOSED GROIN TO BE BUILT WITH WALL SHOWN IN FIG. 1

It it observed that all along the southern coast of New Jersey there are many types of bulkheads, some of wood and some of concrete, and that practically every type has been breached at some point. There are, however, two or three short sections of bulkhead which have a face sloping away from the sea and these appear to have withstood the pounding of the waves better than sections with a vertical face. It was also noted that there is little protection for the back of the bulkheads in the way of tight platforms and that water rushing over the bulkheads in storms readily reaches their back and causes rather early destruction. These observations were utilized in the design noted here.

The design consists essentially of a heavy timber bulkhead running along the shore line with occasional right-angled groins extending out into the ocean, the bulkheads serving to resist the attacks of the water and the groins to prevent erosion of the foreshore. The design of the bulkhead is shown in detail in Fig. 1. It will be noticed that it has a face inclined to the vertical and that certain of the waling pieces have been placed on the outside so as to break up the receding waves and thus minimize the scour. A tight platform has been planned behind the top of the bulkhead in order to protect its back from the water. According to the designer it would be most desirable to give this bulkhead weight by piling in it large quantities of riprap and heavy stone, but in the particular places for which it was designed the cost of stone is very great and the same results have been sought by a relieving platform. This platform has been added near the bottom of the bulkhead on which the sand is to be piled, and thus adds its weight to the bulkhead to aid in preventing its overturning. Such a relieving platform is, of course, common in some kinds of retaining-wall and bulkhead construction, but has not generally been used for sea bulkheads. In order to further safeguard against the possibility of underscour, sheetpiling is recommended from the toe of the apron to a depth of 8 ft.

Cross bulkheads are indicated to be placed every 24 ft. and on the ends of the main bulkhead, so that in case the main line is breached at any point sand will be retained on the relieving platform except for the particular section which is breached.

Fig. 2 shows the design of the groins. These are attached to the bulkhead and extend out into the ocean about every 300 ft. They are to be built 2 ft. above the level of the beach as it now exists, following the so-called "Case" system of low groins common in British

practice. Provision is made so that they may be added to as the beach rises. The planks that are added to increase the height of the groins are comparatively light, so that in the event of the beach being eroded to any appreciable extent during heavy storms the planking will be broken and destroyed before the pile supports, thus relieving the strain on the latter. It will be noted that the inshore structure of the bulkhead serves as a boardwalk.

Removing and Utilizing the Water Hyacinth

The water hyacinth, which was originally introduced in the St. Johns River in Florida as an ornamental plant, has spread throughout that river and other Southern rivers where it was later introduced, forming an obstruction to navigation so serious that for the past fifteen years Federal engineers have had to undertake the removal of the floating masses in order to keep the rivers open to navigation.

The first plan proposed for removing the hyacinth was the equipment of a boat to raise and crush the plants. This proved to be impracticable, and in 1902 spraying the plants with an arsenical compound was adopted. About $85,000 has been expended in clearing the St. Johns River and its tributaries. The poisoning of cattle which feed upon the hyacinth caused the enactment of a law in Florida in 1905 prohibiting the use of poison in destroying the plants. The plan then adopted was to break up the floating masses with a suitable boat and haul them into the current where they would float away. In 1909 a hyacinth elevator was built, consisting of a scow equipped with an inclined conveyor driven by a gasoline engine. With this scow the plants are either piled on shore or deposited in masses in the central current where they are carried away.

In the waters of Louisiana and Texas the plants are still killed by poison. The spraying solution used is made by dissolving one pound of white arsenic and one pound of salsoda in a gallon of water by boiling for about an hour. This stock solution is diluted with about 17 gal. of water per gallon for spraying. One gallon of the diluted solution will kill 10 sq.yd. of the hyacinth. About $200,000 has been expended in fighting the hyacinth, in Louisiana and Texas, and about $20,000 per annum is required to keep the streams clear.

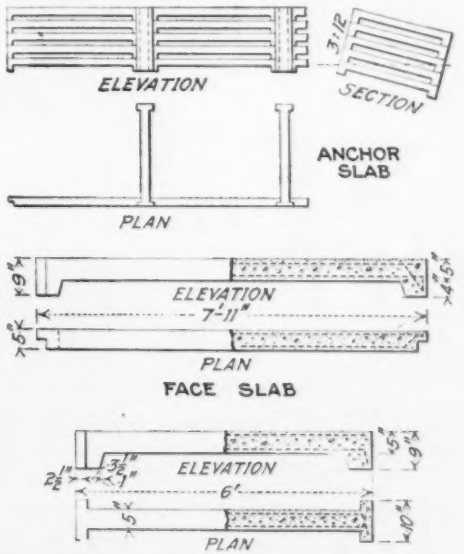
Engineers who are familiar with the rapid and luxuriant growth of the hyacinth in Southern waters will learn with interest that in French Indo-China experiments have been made looking to the utilization of the plant for commercial purposes. While cattle eat the hyacinth greedily, the plants contain so large a proportion of water that the animals lose flesh while feeding on it. According to *The Engineer*, of London, however, the fiber of the water hyacinth has been used in French Indo-China in the manufacture of rope, twine, matting, boxes, chairs, cradles and other articles of furniture, and has even been mixed with silk to form a somewhat stiff but durable cloth. In China, as on the Gulf coast, masses of the hyacinth have choked the waterways and interrupted navigation.

The Engineer further states that since April, 1911, the hyacinth growing in the tributaries of the upper Nile has been taken from the water, dried in the sun, pressed into briquettes and used as fuel at Khartum and other points on the upper Nile. Any American manufacturer desiring to utilize the hyacinth growth in the Gulf-coast waters will be assured in advance of the hearty coöperation of the Federal engineers.

Concrete Crib with Precast Members

The accompanying drawing shows the details of reinforced-concrete cribs used by the Chicago & Western Indiana R.R. to hold the slope of banks. It has proved more economical than a retaining wall and should be more durable than the common timber crib, which it imitates in form.

The face members are about 8 ft. long and have the ends recessed to receive the T-heads of anchor members 6 ft. long. Both face and anchor units are 5x5 in. in section, with ends 9 in. deep, giving 4-in. vertical openings between units. It is thought, however, that in future work it will be an advantage to reduce the width of these openings. The concrete is a 1:2:4 mix, made with gravel or crushed stone that will pass a 1-in. ring. The face units have 0.054 cu.yd. of concrete and 32 ft. 8 in. of ¼-in. square steel reinforcing bars; the anchor slabs have



Concrete Cribbing Used on the Chicago & Western Indiana R.R.

0.047 cu.yd. of concrete and 26 ft. 10 in. of steel bars. The weights are 218 lb. and 190 lb. respectively.

This cribbing was designed primarily as a substitute for retaining walls in certain locations where the height above ground would not exceed 8 to 10 ft. It is placed so as to give a heavy batter on the face (3-in. per ft.) in order that any tendency to slide may be resisted by the vertical rise of the members. Among the advantages of this construction are (1) a reduction of 75 to 80% in cost as compared with retaining walls, (2) the possibility of easy adjustment in case the height needs to be changed, and (3) permanence as compared with timber cribbing. For information we are indebted to E. H. Lee, Vice-President and Chief Engineer of the Chicago & Western Indiana R.R.

News of the Engineering World

Bridge-Floor Fire Destroys Detroit-Belle Isle Bridge

On Apr. 27 a fire in the creosoted-wood floor system of the iron truss bridge between the mainland at Detroit, Mich., and the city's park of Belle Isle completely wrecked ten spans and half of the drawspan. As shown in the accompanying views, most of these spans were so buckled and weakened as to cause their collapse into the river. The bridge, completed in 1889 at a cost of $325,000, was made up of 12 wrought-iron through-truss pin-connected spans, 156 ft. each, a 318-ft. swingspan and pony-truss approaches. Its total length was 2543.6 ft.

The floor system consisted originally of wooden stringers and plank flooring, but in 1906 this was changed to heavier stringers supporting 4-in. creosoted planks, on which were laid 3½-in. creosoted blocks across the 24-ft. roadway. Plank walks 9 ft. wide were supported on brackets outside the trusses.

During a stiff southerly wind, on the afternoon of Apr.

FIRE DESTROYS DETROIT-BELLE ISLE BRIDGE

(1, Two spans and part of drawbridge span at Detroit end still standing. 2, Looking over burned spans toward island. 3, Fallen south end of drawspan. 4, Bare piers, looking toward island; single remaining span in distance. 5, Looking through drawspan toward island, showing type of floor.)

27, a city tar-wagon passing from the island to the mainland probably dropped hot cinders from its ashpan, which must have landed in the cracks in the wooden curbing in several places, as the fire was first discovered under the drawspan and next under the island end of the main through spans. The floor burned very quickly. Attempts to extinguish the fire by bucket were futile. When the fire was first seen there were several motor cars and many pedestrians on the bridge. So rapid was the spread of the conflagration that a number of persons had narrow escapes from between the two evident fires. An effort was made to fight the fire from fireboats, but little progress was made, mainly on account of the difficulty of reaching it through the dense smoke from the creosoted wood. The shore apparatus managed to save the two spans nearest the mainland.

Trusses 1 and 2 at the Detroit end are still on their piers. The adjacent half of the drawspan is in position, but in very bad condition. The south half of the draw buckled at the center and fell into the river. The three short spans at the island are also in position, as is one of the 156-ft. trusses near that end.

Most of the steelwork is at present submerged, but the few broken ends of lower-chord members exposed show a reduced cross-section and considerable elongation, indicating that more heat was generated than one would expect to find in such a fire. The piers are built of good-grade limestone, and the amount of spalling and cracks near the top give further evidence of the heat generated.

The city is now making arrangements for the immediate building of a pile bridge at a temporary site, to be finished June 15, and at a special election the bond issue for a new bridge will doubtless be voted. At the April election a bond issue to replace the old bridge or to build a new one of greater capacity further up river failed to carry. A design for this new bridge, to be of reinforced-concrete arches, has been made by Barclay Parsons & Klapp, of New York City.

Waterproofing the Eden Park Reservoir, Cincinnati, Ohio

A contract for supplying waterproofing material for continuing the efforts to make the Eden Park reservoir at Cincinnati, Ohio, water-tight has been signed between the city and the Minwax Co., Inc., 18 E. 41st St., New York City. The reservoir is an old one. A few years ago leaks were disclosed and a lining was put in by contract. This did not prove to be tight and later repairs have not made the job satisfactory. The exact method of applying the new waterproofing material has not yet been made public. J. A. Hiller is General Superintendent of Water-Works at Cincinnati and J. B. Gardner is Secretary of the Minwax Co.

The Chicago Union Station

Preliminary work has been commenced for the construction of the new Union Station at Chicago, to take the place of the old station at Adams and Canal St., now used by five of the railways entering the city. The main passenger building, 300x400 ft., will occupy the block bounded by Adams, Jackson, Canal and Clinton St., a subway 100 ft. wide running east under Canal St. to a train concourse 175x250 ft. extending between two groups of stub tracks. There will probably be 10 tracks for lines at the north end and 15 for those at the south end of the station. A pair of through tracks will provide for interchange or transfer movements, and there will be extra tracks for baggage, mail and express service. The track level will be lowered slightly, being kept high enough above the water level of the river to insure sufficient drainage. The cost of the terminal, including land and construction, is estimated at about $60,000,000.

The new freight terminals will extend from Harrison St. south to 16th St., and will occupy most of the space between Canal St. and the river. The Chicago, Burlington & Quincy freight terminal will front on Canal St. (with a freight house between Harrison and Taylor St.), and the Pennsylvania Co.'s terminal will be between this and the river. The latter company will build a freight house fronting on Stewart Ave. and extending from Polk to Taylor St. The freight tracks will be about 4 ft. below the present level (which is approximately at street level). The streets will be raised to about the level of the bridges over the river, and Canal St. (forming the western boundary of the site) will be raised to the same level.

In planning its new terminal the Pennsylvania Co. originally proposed an independent freight terminal between Van Buren, Polk, Jefferson and DesPlaines St., west of the passenger station. This was to be approached by an elevated branch line, having inclines to reach surface tracks (as well as the upper tracks) in the terminal. This project was strongly opposed by the city as unnecessarily extending the already excessive area occupied by terminals and forming an additional barrier to the westward growth of the business district. The opposition was so strong that the railway company abandoned the project.

For the new passenger station and terminal the Union Station Co. has been organized, representing jointly the Pennsylvania Co. (in the majority), the Chicago, Burlington & Quincy R.R., and the Chicago, Milwaukee & St. Paul Ry. The Chicago & Alton Ry., which uses the present station and will be a tenant of the new one, is not included in the owning company.

The construction is in the hands of the Pennsylvania Co., under Thomas Rodd, Chief Engineer of the Pennsylvania Lines (Pittsburgh, Penn.) and also Chief Engineer of the Union Station Co. J. D'Esposito, Assistant Engineer, Pennsylvania Lines, is in general charge of the work. The architects are Graham & Burnham, of Chicago. The Pennsylvania Co. has let two contracts—one for clearing the site for its new freight house, to P. T. Clifford & Sons, of Valparaiso, Ind.; the other for the construction of the temporary freight house, to the Sumner Sollitt Co., of Chicago.

Fire Sweeps Colon, C. Z.

A fire on Apr. 30, starting in the telephone exchange of Colon, C. Z., was swept by a high wind toward the Cristobal section of the city. Here 12 buildings were dynamited in an effort to stay the flames. The conflagration cost the lives of 11 persons (no Americans) and 7000 persons were rendered homeless. The property damage is reported to exceed $3,500,000. The United States Commissary depot—the largest building on the Isthmus—was destroyed, as were also the buildings of the Panama Banking Co. and the International Banking Corporation.

Progress on 46th St. Pier Coffer-Dam, New York City

The coffer-dam at the site of the proposed new piers at the foot of 46th St. and the North River, New York City, is now practically completed, and pumping has been in progress for some weeks. This coffer-dam, described in ENGINEERING NEWS, July 23, 1914, p. 212, when emptied will be the deepest ever built. It is made up of continuous cells of steel sheetpiling, each filled with earth and rock, and the whole line is backed inside and outside with a heavy riprap fill, forming in effect a rock-fill dam with a core wall of steel cylinders. The area to be unwatered will be about 800 ft. in length and 300 ft. in width, with a final head of nearly 65 ft.

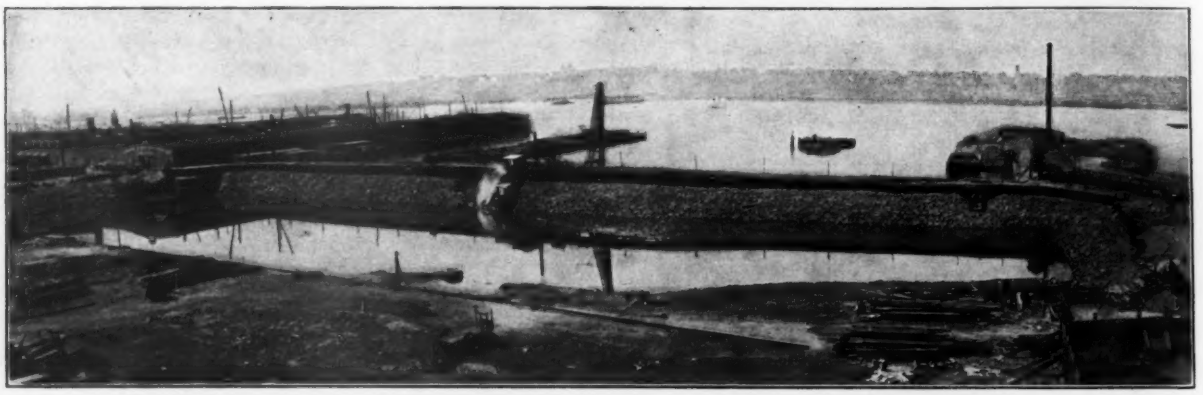


FIG. 1. COFFER-DAM IN 46TH ST. PIER PARTLY UNWATERED; HEAD ABOUT 13 FT.

The pumping was started with a 10-in. pump, which, with a capacity of about 250 gal. per min., reduced the head at a rate of 6 or 7 in. per day of 16 hr. This reduction continued until the level inside the dam was 13 ft. below low water, giving a 17½-ft. head at high water. At this point leakage developed under the single row of sheetpiles driven on the 47th St. or north side of the coffer-dam. Here an old crib prevented the use of the cylinders and a single line of steel sheetpiling had been driven and tied back to the old cribbing. Evidently leakage was taking place between the sheetpiling and rock.

A curious way of stopping the leak was tried. At first it was thought that it would be better to grout, but the expense precluded this method. Instead a 12-in. round timber pile was driven in the old fill back of the crib and the pile then pulled. The opening was then filled with layers of earth and sawdust and manure. The plugs of this material were driven down until the whole plug had been driven into the subsurface and the pile was in its old position. The pile was then drawn and the process repeated. This was done from three to a dozen times in holes at intervals of two feet.

FIG. 2. LOOKING ALONG 46TH ST. COFFER-DAM; HUDSON RIVER AT LEFT, UNWATERED AREA AT RIGHT

The leak lasted a week and was of such extent that the 10-in. pump was able to keep a level; that is, from 6 to 7 in. per day apparently leaked into the dam. At last the leak was plugged and pumping continued until a week ago, when at a level of 15 ft. another leak of the same sort developed. The plugging work, as described in the foregoing, was continued, and on May 3 no leakage was apparent and the level was down to —21 or 25 ft. below high water. At present the one 10-in. pump is working 16 hr. per day from a platform on the dam and two 6-in. and one 4-in. pumps are working 8 hr. per day on a raft inside the dam. No effort is being made to reduce the water level very fast, because it is thought better to let the dam take a set at intervals.

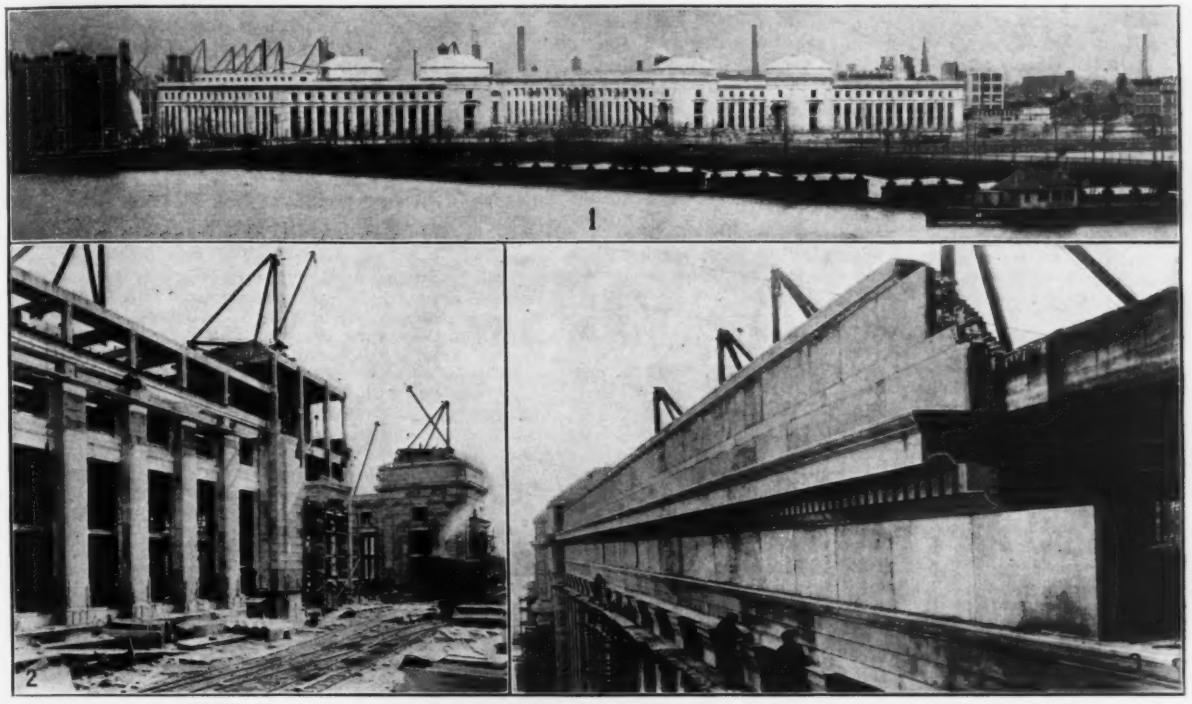
The work is under the direction of the Department of Docks and Ferries of the City of New York, of which R. A. C. Smith is Commissioner of Docks and Ferries and C. W. Staniford, Chief Engineer. Holbrook, Cabot & Rollins have the contract for all of the work.

Philadelphia Rapid-Transit Extensions Assured

By a vote of 85,992 to 8772 the voters of the city of Philadelphia, in a special referendum held Apr. 29, authorized City Councils to borrow $6,000,000 to begin the construction of the Broad St. subway and the Frankford elevated railway. The exact routes of the lines to be constructed must now be determined by City Councils in an authorizing ordinance. The Department of City Transit has the plans for the work in an advanced state.

Progress on New Technology Buildings

Recent progress in the construction of the new buildings for the Massachusetts Institute of Technology at Cambridge allows a better idea than before of their final appearance. The simplicity of outline of the connected structures at a distance is shown by Fig. 1 in the accompanying views. This view is still flat, since it lacks the great dome of the library rising to about three times the present height and changing the skyline. The other views show the structures more in detail and are of particular interest because of a very pretentious exterior carried by a reinforced-concrete frame. The standard-gage tracks shown have been laid around the work to such an extent that materials may be unloaded at the point of use without the expense of transshipment or cartage. The plans of the "New Technology" were described in ENGINEERING NEWS, Nov. 20, 1913.



PROGRESS VIEWS OF THE NEW TECHNOLOGY BUILDINGS

(1, Educational group from across Charles River; 2, Chemistry and General Studies Buildings; 3, Cornice and parapet wall of Mechanical Engineering Building)

Large Sewer-Tunnel Completed

The Mile Rock tunnel, through which the entire storm flow from the sewers of the Western drainage area of San Francisco will be carried to the ocean, is practically completed. Crews working from both ends met on Apr. 11, 1915, 2850 ft. from the south portal. The error in grade and alignment at the point of meeting was less than 0.016 ft. The tunnel is lined with concrete, is 9x11 ft. in cross-section and 4450 ft. long. Only storm water will flow through the tunnel, all domestic sewage being pumped, during the dry season, to the Baker's Beach outfall sewer, now being constructed.

A Premature Blast on Apr. 27 in a cutoff trench at the new Kensico Dam of the New York City Water Supply killed seven Italian workmen and injured 11. Three holes had been loaded and the electric-fuse leads brought forward, but connected to only one side of the firing circuit. The remaining lead was lying on the ground. The workmen were leaving the trench and a foreman was instructing a man how to use the firing switch when the charges exploded. Attempts since to fire charges under like conditions have given negative results. The cause is unexplained.

More Floods in Texas—On Apr. 27 floods occurred in Texas which, while not as severe as were those of December, 1913, caused more damage than the floods of May, 1914. In the valley of the Brazos, people were compelled to vacate the lowlands. An authoritative report, dated Apr. 30, states that the Brazos and Trinity Rivers were well out of banks on that date. The crest of the Brazos had just left Hempstead; the lower portion of the stream will remain above flood stage until May 8, at least. The Trinity was still flooded from Dallas southward nearly to Trinity County. The flood was moving slowly and it will probably be three weeks before it passes off into the Gulf. The Colorado was still flooded from Columbus southward to its mouth.

The Population of the United States passed the 100,000,000 mark on April 8, according to the U. S. Census Bureau. This result is obtained by adding to the ascertained population of Apr. 15, 1910, increments covering the intervening period at the rate established by the difference between the census of 1900 and that of 1910, that is, 134,828 per month.

Transfers with a Jitney-Bus Line will be exchanged between the San Francisco municipal railway and a bus line operating across Golden Gate Park and on Great Highway along the ocean beach. This is intended to provide more adequate transportation facilities for a district now without railways. Accurate cost data will be kept, and if the bus line proves possible of profitable operation it will be taken over by the city.

Rear-Wheel Fenders for Motor-Busses have been adopted by the Fifth Avenue Coach Co., in New York City. The design, as approved by the Public Service Commission, provides for attachment to the vehicle sides in front of the rear wheels. It has been found that accidents in front can be sufficiently guarded against by the drivers, but a number of persons have been injured by the rear wheels without the knowledge of the operators.

The Adamson-Shields Bill for the lease of rights to water-power development on public lands is to be reintroduced at the next session of Congress, according to an announcement by Secretary Lane on Apr. 28. The bill, which was discussed editorially in our issue of Jan. 21, p. 131, passed the House in the last Congress, but action upon it was delayed in the Senate until so late in the session that its passage was only possible during the closing hours by unanimous consent. Objection by a single senator caused the failure of the bill.

Statistics of Unemployment in New York City in the first half of February, 1915, are made public by the Bureau of Labor. Information obtained from 54,849 families, numbering 229,428 persons, of whom 95,443 were wage earners, showed 11,723 wage earners out of work, or 16.2% of the workers. Comparing this percentage with the population of Greater New York indicates a total of about 400,000 unemployed at that time. These figures do not show the total extent of unemployment, however, since no account is made of wage earners working on part time or employed irregularly. On the other hand, a considerable percentage of the unemployment was due to February's being a slack season in the clothing and building trades. A similar investigation made a few weeks earlier by the agents of the Metropolitan Life Insurance Co. showed by enquiry in 155,960 families that 18% of the wage earners in those families were out of employment.

New Building on Old Astor House Site—Nearly two years ago the southerly half of the historic Astor House, built in 1836 at the junction of Park Row and Broadway, New York City, was torn down to facilitate the construction of the Vesey St.-Broadway "S" curve of the Broadway-Lexington Ave. subway, now under construction. The city, by a payment of $600,000, secured a permanent sub-surface easement under the lot and at the same time the use of the property for a contractor's yard. The subway at this point was arranged so that the columns of any new building to be built would not bear directly on the subway structure, but would be carried through the medium of concrete piers sunk in caissons to rock. It has now been announced that Vincent Astor, the owner of the property, is to erect on the southerly half of the site a seven-story office building, fronting 100 ft. on Broadway and 157 ft. on Vesey St. The northerly half, which is owned by William Waldorf Astor, has not been taken down, nor has any announcement been made providing for its future disposition.

Offers for Motor-Bus Franchises on 25 miles or more of streets in the Borough of Manhattan, New York City, are requested by the Board of Estimate and Apportionment. The franchise drafted by the Board's Franchise Bureau contains strict requirements as to the regularity of operation, the character of vehicle used and the service to be rendered. The companies bidding are to state the sum they are willing to pay for a 15-year franchise, first in cash and second in percentage of annual gross receipts, the rates of fare to be charged, and the minimum headway under which the busses will be operated. The vehicles to be used are limited to a maximum weight of 10,500 lb., a width of 7 ft. 6 in., and a height over all of 12 ft. 6 in. They must not carry more passengers than can be seated, and the companies operating the routes are obliged to keep the streets clear of snow and ice for a width of 16 ft. and to deposit $30,000 in cash with the City Comptroller as security for the performance of all the conditions of the contract. In general, the routes laid down for motor-bus operation are those on which street-railway service is not furnished, as the policy of the Board's Franchise Committee has been to avoid competition of the motor busses with the street railways.

Mr. Edison on the Edison Fire—Thomas A. Edison has recently sent to the technical press and newspapers a letter in which he calls attention to a campaign against concrete which is being waged by various brick interests, taking the Edison fire as the text. According to Mr. Edison the entire purpose of the publication is to discredit and, if possible, retard the use of reinforced concrete in construction of fireproof buildings. Mr. Edison very strongly states that the evident purpose of the publication is to deceive and that it is in error in many ways. His specific criticisms are as follows:

The results of the fire at my plant on Dec. 9, 1914, are used in an entirely false and misleading manner. Of the seven reinforced-concrete buildings, none were destroyed. A small section of the upper floor of one of the buildings fell in, but was supported by the lower floors. The pamphlet referred to presents three views of this, the suggestion being that they were of three different buildings. The brick administration building to which it refers, which remains standing, was protected by an adjacent concrete building and was not subjected to the fire.

Every brick and steel building which was attacked by the fire was completely destroyed, together with all the machinery they contained, while the damage done to the concrete buildings amounted to about 12½ per cent., and of the machinery contained in the concrete buildings 98 per cent. was saved and is now in operation. Manufacturing was resumed in some of the old concrete buildings within a few weeks after the date of the fire.

Temperatures were far in excess of those in the ordinary fire, but reinforced concrete showed its superiority over any other fire-resisting material.

ENGINEERING SCHOOLS

STEVENS INSTITUTE OF TECHNOLOGY—The debt-retirement and new-endowment funds which, as noted in this column, Apr. 29, the alumni have been strenuously working for, were completed on Apr. 30. The total increase in funds as a result of this campaign is $1,380,000, compared with the $1,360,000 required to discharge all debts and meet all conditional gifts, and with the $1,375,000 aimed for. A half-million is now available for improving the Institute's plant, and it is proposed to proceed at once with the gymnasium and the Wm. Hall Walker Laboratory of Mechanic Arts (for shopwork).

PERSONALS

Mr. H. S. Collette, Secretary of J. G. White & Co., Inc. and the J. G. White Engineering Corporation, has resigned from these companies, and expects to reside permanently in California.

Mr. James O. Carr, General Manager of the General Electric Co., Schenectady, N. Y., has been nominated by Governor Whitman, of New York, to succeed Mr. Martin S. Decker as a member of the Public Service Commission for the Second District.

Prof. W. R. Eckart, M. Am. Soc. M. E., of Leland Stanford, Jr., University, has been appointed by the Institute of Mechanical Engineers of London to represent that society at the International Engineering Congress in San Francisco in September.

Mr. John Wilson, formerly City Engineer, of Duluth, Minn., has resigned and will open an office at 300 First National Bank Building, Duluth, Minn., where he will carry on a general practice in civil and sanitary engineering, making a specialty of municipal problems.

Mr. Gaylord C. Cummin, Assoc. M. Am. Soc. C. E., City Engineer of Dayton, Ohio, has been made City Manager of Jackson, Mich. Mr. Cummin was graduated in civil engineering from Cornell University in 1904 and has been City Engineer of Dayton for a number of years. He held that office at the time of the destructive floods of 1913.

Mr. John Anderson, formerly General Superintendent for George B. Spearin, and Mr. James Wheeler, recently with Westinghouse Church Kerr & Co., have formed a company to be known as Anderson & Wheeler, Inc., which will take up work as contracting engineers on dock, timber and concrete construction. The offices of the company will be at West New Brighton, Staten Island, N. Y.

Mr. Clinton White has announced that he will retire from the Public Service Commission of Massachusetts on May 18, his 70th birthday. Mr. White was a member of the Special Commission on Docks and Terminal Facilities from 1895 to 1897, of the Harbor and Land Commission from 1897 to 1901, the Railroad Commission and its successor, the Public Service Commission, from 1901 to the present time.

Mr. J. A. L. Waddell, M. Am. Soc. C. E., has had conferred upon him by the Japanese Government, through its Department of Education, the degree of Hakushi (Doctor of Engineering). This is the fourth doctor's degree conferred on Dr. Waddell, the three other degrees being Doctor of Science, McGill University, 1904; Doctor of Laws, Missouri State University, 1904, and Doctor of Engineering, University of Nebraska, 1911.

Mr. H. A. Macbeth, M. Am. Soc. M. E., has been appointed Superintendent of Motive Power of the New York, Chicago & St. Louis Ry., succeeding Eli A. Miller, whose death was announced in last week's issue. Mr. Macbeth entered the service of the Nickel Plate in 1883 as an apprentice and passed through the various grades until 1905, when he became Master Mechanic of the shops at Conneaut, which position he held at the time of his present appointment.

Mr. Louis K. Rourke, M. Am. Soc. C. E., resigned from his position as Commissioner of Public Works of the City of Boston, Mass., at the end of his term on Apr. 30. Mr. Rourke's plans for the future have not been definitely made, but it is announced that he is intending entering the private practice of engineering. Building Commissioner Patrick O'Hearn has been designated by Mayor Curley as Acting Commissioner of Public Works until an appointment to the position is made. The office pays a salary of $9000 a year.

Mr. Frederick H. Newell, M. Am. Soc. C. E., has been appointed Professor of Civil Engineering and head of the Department of Civil Engineering at the University of Illinois. Mr. Newell was born in Pennsylvania in 1862 and was graduated in mining engineering from the Massachusetts Institute of Technology in 1885. After some preliminary experience in mining in Colorado, Ohio, Pennsylvania and Virginia he entered the United States Geological Survey as Assistant Hydraulic Engineer in 1888. He rose through the various grades in that service until 1902, when he was appointed Chief Engineer, and, in 1907, Director, of the United States Reclamation Service, which was formed in 1902. Mr. Newell resigned as Director of the Service early this year and became Consulting Engineer, which position presumably he will retain. Mr. Newell expects to take up his work at the University of Illinois immediately.

OBITUARY

DeWitt C. Stephens, general contractor, of Buffalo, N. Y. (and with offices at Des Moines, Iowa), died in New York City on Apr. 21. Mr. Stephens made a specialty of large drainage contracts, and at the time of his death had in hand a $1,250,000 contract near Cape Girardeau, Mo.

Charles W. Blakeslee, Jr., a member of the contracting firm of C. W. Blakeslee & Sons, New Haven, Conn., died suddenly at his home in New Haven on Apr. 28. Mr. Blakeslee was in his 71st year, and was associated with his father and brothers in a contracting firm which has not only done a great amount of work in New England, but also had one of the big sections in the new Catskill Aqueduct for the City of New York.

Col. T. H. Handbury, M. Am. Soc. C. E., Corps of Engineers, U. S. A., retired, died in Berne, Switzerland, on Apr. 23, at the age of 74. Col. Handbury was graduated from West Point in 1861 and was transferred to the Corps of Engineers in 1866. His professional work included services in the Pacific and Mississippi Districts and a four-year term as Professor of Engineering at West Point. He was retired in 1895.

Eldredge Dale, an engineer who for many years was connected with the Pennsylvania R.R. and the Philadelphia & Reading Ry., died at his home in Reading, Penn., on Apr. 27, at the age of 73. Mr. Dale's early service was with the Pennsylvania R.R., but he was later connected with the construction of the Sixth Ave. Elevated Line in New York, the Reading Terminal in Philadelphia and the Reading work around Reading, Penn.

George A. Perkins, formerly Chairman of the Massachusetts Highway Commission and leader in the good-roads movement, died at Boston on Apr. 29 at the age of 59. His profession was the law, but as an enthusiastic bicyclist he became interested in roads, and in 1887, when a member of the Massachusetts Legislature, he introduced the first bill for the improvement of the state highways. The idea of a State Highway Commission originated with him and he introduced the bill establishing it in his state.

Charles F. Spaulding, consulting engineer to the Newton & Watertown Gas Co., formerly president of the New England Gas Engineers Association and of the Guild of Gas Engineers, died at his home in Waltham, Mass., on Apr. 28 at the age of 87. He was a native of Franklin, Vt., and was at various times superintendent and engineer of gas plants at Burlington, Vt., and Brookline and Waltham, Mass. He served in the Civil War as a Major in the 16th Vermont Volunteers and after the war was a Colonel in the Vermont Militia.

William M. Foote, President of the Commonwealth Power Co., died at his home in Jackson, Mich., Apr. 14, aged 61 years. Mr. Foote had been connected with various electric and hydro-electric plants in the State of Michigan since the earliest days of the industry, and of late years had devoted considerable attention to the building and operating of interurban electric roads. A Michigan engineer has said that "Mr. Foote did more for the state than any other man in developing its natural resources. His foresight, energy and courage inspired his associates and subordinates year after year, and his patience, his democratic spirit, his tolerance, his cheerfulness and his kindliness endeared him to anyone he met in any capacity."

ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN WATER WORKS ASSOCIATION.
May 10-14. Annual meeting in Cincinnati. Secy., J. M. Diven, 47 State St., Troy, N. Y.

NATIONAL FIRE PROTECTION ASSOCIATION.
May 11-13. Annual meeting at Hotel Astor, New York City. Secy., Franklin H. Wentworth, 87 Milk St., Boston, Mass.

CONFERENCE OF BUILDING INSPECTORS.
May 14. Meeting in New York City. Secy., Sidney J. Williams, Madison, Wis.

INTERNATIONAL RAILWAY FUEL ASSOCIATION.
May 17-20. Annual meeting at Chicago. Secy., C. G. Hall, 922 McCormick Bldg., Chicago.

SOUTHWESTERN ELECTRICAL AND GAS ASSOCIATION.
May 19. Annual convention at Galveston, Tex. Secy., H. S. Cooper, Dallas, Tex.

NATIONAL MACHINE TOOL BUILDERS' ASSOCIATION.
May 20-21. Spring meeting at Atlantic City, at Hotel Marlborough-Blenheim. Secy., Charles E. Hildreth, Worcester, Mass.

MASTER BOILER MAKERS' ASSOCIATION.
May 25-28. Annual meeting in Chicago. Secy., H. D. Vought, 95 Liberty St., New York City.

AMERICAN SUPPLY AND MACHINERY MANUFACTURERS' ASSOCIATION.
June 3-4. Annual convention at Bellevue-Stratford, Philadelphia, Penn. Secy., F. D. Mitchell, 1510 Woolworth Bldg., New York City.

NATIONAL DISTRICT HEATING ASSOCIATION.
June 1-3. Annual convention at Hotel Sherman, Chicago. Secy., B. L. Gaskill.

NATIONAL CONFERENCE ON CITY PLANNING.
June 7-9. National Conference in Detroit, Mich. Secy., Flavel Shurtleff, 19 Congress St., Boston, Mass.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 9-11. Annual convention in Atlantic City. Secy., Jos. M. Taylor, Karpen Bldg., Chicago.

MASTER CAR BUILDERS' ASSOCIATION.
June 14-16. Annual convention in Atlantic City. Secy., Jos. M. Taylor, Karpen Bldg., Chicago.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
June 17-18. Semi-annual meeting at Toledo. Secy., Frank E. Sanborn, Ohio State University, Columbus.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 22-25. Annual meeting at Iowa State College, Ames, Iowa. Secy., F. L. Bishop, University of Pittsburgh, Pittsburgh, Penn.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
June 22-25. Summer meeting at Buffalo, N. Y. Secy., Calvin W. Rice, 29 W. 39th St., New York City.

AMERICAN SOCIETY FOR TESTING MATERIALS.
June 22-26. Annual meeting in Atlantic City. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia, Penn.

American Electrochemical Society — At the 27th general meeting, at Atlantic City, N. J., Apr. 22-24, the following elections were announced: President, Lawrence Addicks; Vice-Presidents, L. E. Saunders, J. A. Mathews, W. H. Walker; Treasurer, F. G. Salom; Secretary, J. W. Richards.

The Pan-American Road Congress is to be held in Oakland, Calif., Sept. 13-17, under the joint auspices of the American Highway Association and the American Road Builders' Association. Anyone not a member of either association, desiring to become a member of the Congress and receive a copy of the printed proceedings, may join as a temporary member by paying a fee of $2.

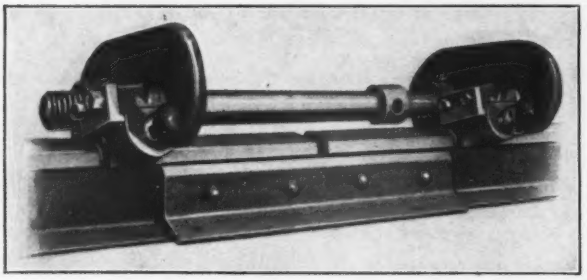
American Society for Testing Materials—The nominating committee has prepared the following ticket for the ensuing year: President, Mansfield Merriman; Vice-President, William H. Elxby; members of Executive Committee, James W. Gibboney, William K. Hatt, John A. Mathews and Edward Orton, Jr. The next annual meeting will be held, June 22-26, at the new Traymore Hotel, Atlantic City, N. J.

National Fire-Protection Association—At the coming 19th annual meeting of the association in New York City, at the Hotel Astor, May 11-13, a number of interesting topics will be under discussion. The regulations for automatic sprinklers will be revised, a complete report on interior standpipe systems will be presented, a complete revision of the regulations for gravity and pressure tanks and revisions of the regulations for blower systems and for rotary and centrifugal fire-pumps will be considered. Following the annual meeting the building commissioners and inspectors in the associate membership of the Society will hold a special conference for the discussion of building topics. The headquarters of the Society is at 87 Milk St., Boston, Mass.

Appliances and Materials

Rail-Joint Opener

In installing signal equipment it is frequently necessary to open rail joints sufficiently to admit of placing the fiber end piece for insulating the joint. To perform this work quickly and without damage the Chicago Railway Signal & Supply Co. has introduced a device which consists of a pair of clamps connected by a threaded bar, as shown in the accompanying view. Each clamp is gripped to the rail head by a



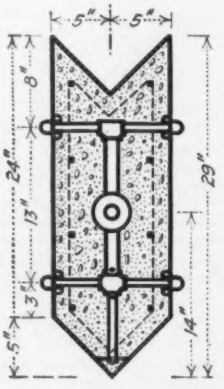
RAIL-JOINT EXPANDER

capstan screw, and by turning the threaded bar by means of its capstan head the clamps and rails are forced apart and held while the insulating end-piece is inserted. It is not necessary to remove the splices or spikes. The device can be used in the same way either to pull or push rails in getting proper adjustment of bolt holes or signal fittings. The machine weighs about 120 lb. and is usually handled by two men, but it can be handled by one man, and he can remove it from the rail in a few seconds.

* * *

Reinforced-Concrete Sheetpile

The Bignell Piling Co., Lincoln, Neb., has put on the market a reinforced-concrete sheetpile with special attachments for jetting, following closely the design of the load piles described in "Engineering News," Oct. 8, 1914, p. 726. These piles have a continuous 4-in. pipe from which at intervals run horizontal pipes that turn upward at the outer



BIGNELL SHEETPILE

edge of the pile section. In addition a 2-in. pipe extends through the 4-in. pipe to the point of the pile where it is fitted with a brass nozzle through which the digging jet is forced. The jets through the horizontal pipes serve to loosen the earth through which the pile is being sunk. The sheetpile section is notched as shown, to form joints, the precise shape and size of the notch being varied as conditions warrant. The accompanying view shows two sections being driven; the drawing gives a typical section.

* * *

Small Refrigerating Machines

A small, compact automatic refrigerating machine intended particularly for the iceboxes of small factories, dwellings, etc., has been developed by the Montclair Refrigerating Corporation, Woolworth Bldg., New York City. The apparatus comprises a motor-driven rotary-pump compressor (for ethyl chloride) submerged in glycerin to seal and lubricate it, a water-cooled condenser and an expansion coil in a small brine tank for making a small quantity of ice in addition to cooling the icebox. A thermostatic controller turns on and off the electric current and water needed to operate the machine, so that it is entirely automatic. Two sizes are available—"B," having a capacity equal to 100 lb. of ice per day and driven by a ¼-hp. motor, and "C," having a capacity equal to 300 lb. of ice and driven by a ½-hp. motor. The prices are $225 and $325 respectively.

* * *

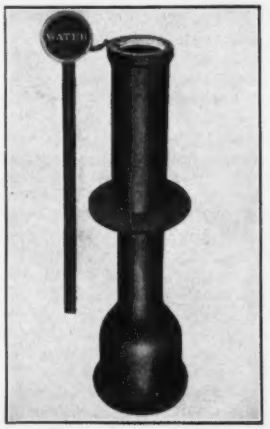
Steam-Flow Meter

An English design of recording steam-flow meter (also for gases and liquids generally), the "Curnon" type, is being made in America by J. G. Biddle, instrument maker, of Philadelphia. It comprises essentially a Pitot tube and differential pressure gage. The latter is a flat box separated by a diaphragm into two chambers which are connected to the upstream and downstream nozzles of the Pitot tube. Movements of the diaphragm (necessarily small and oscillating) are transmitted outside the gage by a spindle in a stuffing-box and are multiplied by linkwork leading to the chart-pen arm. A Bourdon tube is introduced into the linkwork to correct for pressure changes, so that true weight is recorded.

* * *

Telescopic Valve Box

A valve box brought out very recently is of a sliding telescopic instead of the familiar screw type, and is so made that the upper section has no bearing upon the lower section. This prevents a heavy load on the top of the box from being transmitted to the valve or the main and so causing leaks or breaks. A bonnet on the lower section holds the valve in position within it, and will fit valves of 4- to 12-in. size. The upper section has a broad flange to take the vertical load due to vehicles, etc., and is fitted with a locking lid, attached by a chain so that it cannot be lost. This box is made by the H. W. Clark Co., Mattoon, Ill. The same company makes a special locking lid (as used for meter boxes) for application to old valve boxes and service boxes, the covers of which have been broken or lost, so that the fittings are being covered up with dirt which enters unhindered.



CLARK VALVE BOX

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Colored Boiler-Gage Liquid

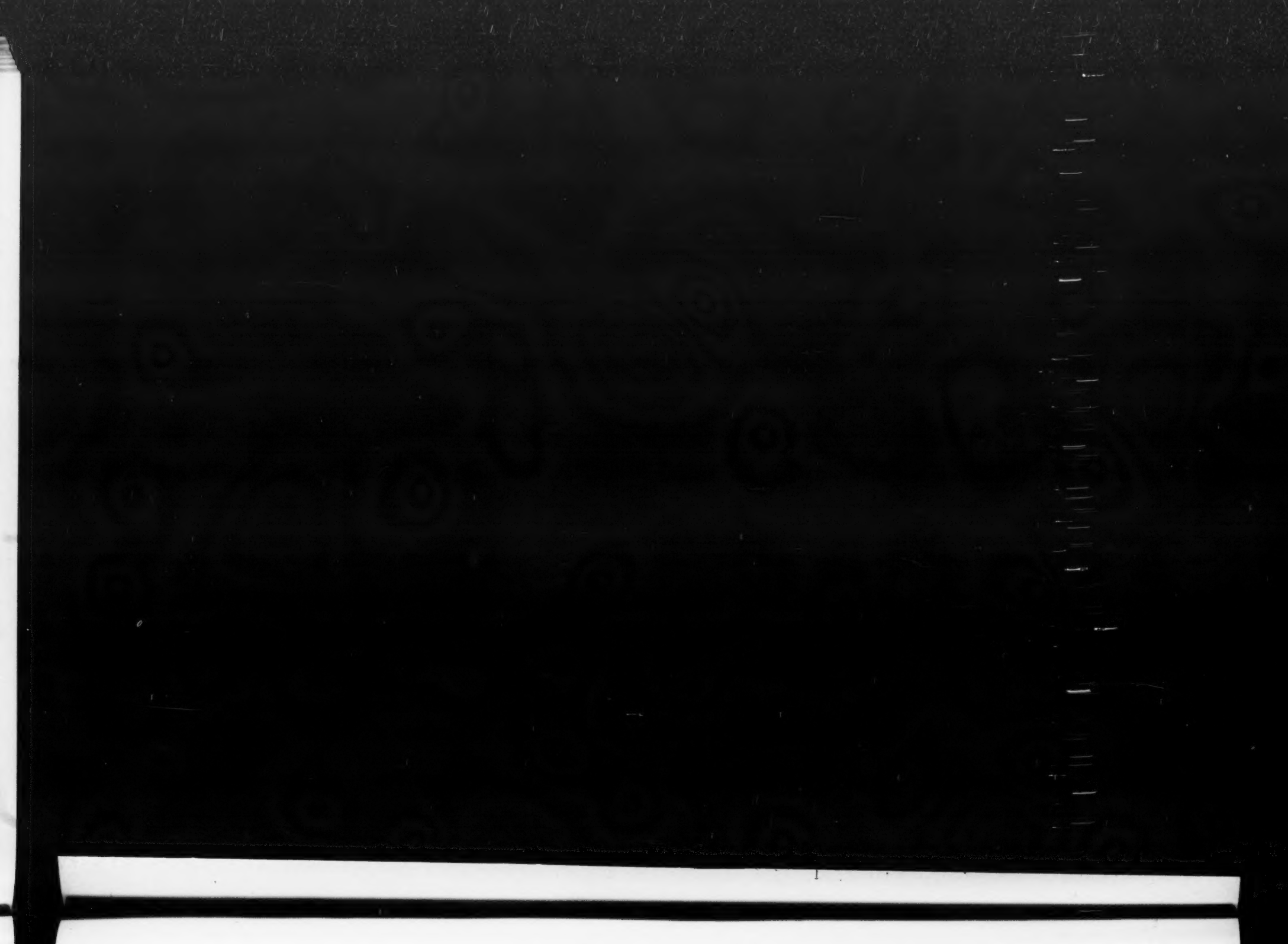
An attachment for steam-boiler gages to color the gage water red is being introduced by Ronald Trist & Co., 4 Lloyds Ave., London, England. The advantage of colored gage liquid is, of course, the ability to see the water level from a distance and at a sharp angle. This attachment is a gun-metal bulb, screwed into the place of the upper gage plug, holding a charge of soluble coloring matter. Steam condenses in this chamber and enough water trickles down to keep the gage column visible. When the color becomes too faint, through repeated blowing down, etc., it is only necessary to add a fresh charge of color to the bulb.

* * *

Small Single-Phase Motors

A new line of single-phase industrial motors, in ½- to 10-hp. capacities and for 110 and 220 volts, has been developed by the Westinghouse Electric & Manufacturing Co., East Pittsburgh, Penn. Above 2 hp. a steel "frameless" design is employed; below 2 hp. a cast-iron frame holds the stator parts. These motors start as repulsion units with large torque and limited current; at full speed an internal mechanical governor automatically changes the connections.

* * *

Small Portable Crusher Plant

A new portable crushing rig has been developed by the Smith Engineering Works, of Milwaukee, Wis., for use on small isolated concrete jobs where good gravel is not easily available but field stone is. There are a jaw crusher (7x10-in. opening and 2 to 4 tons per hour capacity) driven by a 12-b.hp. two-cylinder gasoline engine, a reciprocating feeder, single-chain elevator, chute or rotary screen and five-ton bin. All are mounted on a steel truck, made of 8-in. braced channels, and fitted with 24x6-in. wheels in the front and 30x6-in. in the rear. The rig weighs about 9000 lb. and can be hauled over paved roads by two horses.